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K. M. ALEXANDER presents and analyzes data from approximately 1000 determinations on the "Strength of the Cement Aggregate Bond." On the basis of these data, explanations are suggested for some aspects of the process of tensile failure of plain concrete.

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JAMES R. LIBBY offers a critical study of the three current methods of varying the prestressing moment in prismatic pretensioned beams. Conclusions are made regarding the applicability of each of the methods.

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The structural design and details of the prestressed concrete thin shell cantilever roofs over the grandstands at the new Hipodromo Nacional (National Race Track) now under construction in Caracas, Venezuela, are described by HENRY M. LAYNE and T. Y. LIN in their paper, "Prestressed Concrete Shells for Grandstand Roofs."

On the Cover—Pennsylvania Avenue Bridge over the Canadian River, Oklahoma City. Precast, prestressed girders carry two 28-ft traffic lanes, a 4-ft median, and two 5-ft sidewalks; over-all length, 760 ft. Precast 16-in. piles with cast-in-place bents form the substructure.

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Ultimate Loads and Deflections from Limit Design of Continuous Structural Concrete

By G. C. ERNST and A. R. RIVELAND

Ultimate loads and deflections permissible in individual members of a continuous concrete structure are dependent on the moment distributions. One of the methods used to determine this distribution is the limit design or plastic analysis. This paper gives a brief presentation of the pertinent features of the plastic theory. It defines essential terms and describes the static, kinematic, and uniqueness theorems which are used in conjunction with the analysis. It also discusses three methods used in determining moment distribution and gives accompanying examples and equations. This is followed by comparisons of concentrated plastic rotations and deflections for certain specific cases.

■ INSOFAR AS ACI 318-56 IS CONCERNED, it is assumed that the theory of elastic frames will be used in the calculation of bending moments for the ultimate design of the members of a continuous beam or frame. It is stated that with the specified load factors, stresses under service loads will remain within safe limits. This procedure is a form of limit design in which the distribution of moments is determined by an elastic analysis based on assumed flexural stiffnesses for the members. On the other hand, the customary procedure in limit design is to use the distribution of moments from a plastic analysis.[1]

A given loading and load factors will produce a structure of the same ultimate load capacity by either of the above design methods. This is true because all redundancies are fully developed for both cases, in the first by approximately simultaneous attainment of the ultimate strength at all critical sections and in the second by plastic yielding of all redundant moments until the last critical section has been developed to its full ultimate strength. As a matter of fact, the ultimate load in each case is attained by the utilization of the plastic properties of the steel and the concrete, but to different degrees. In general, the required concentrated plastic rotations and the ultimate deflections will not be the same for equal ultimate loads.

G. C. ERNST, professor of civil engineering and director of the engineering experiment station, University of Nebraska, Lincoln, first became interested in limit design in 1929. In 1940 he conducted research on the limit design of reinforced concrete frames. He has written several papers on the subject. Professor Ernst is a long-time ACI member and currently active on Committee 115, Research, and chairman of Committee 338, Torsion.

●

A. R. RIVELAND is assistant professor of civil engineering, University Nebraska, Lincoln. Prior to joining the Nebraska faculty, he worked for a consulting engineer in North Dakota. Professor Riveland, who joined the Institute in 1956, has engaged in concrete research on ultimate strength in addition to teaching. Currently he is working on research irto the required concentrated plastic angle changes for continuous reinforced concrete structures.

In view of these considerations, a brief presentation of the pertinent features of plastic theory is followed by comparisons of the concentrated plastic rotations and deflections for certain specific cases.

PLASTIC THEORY

Definitions

A *plastic resisting moment* (M_p) remains constant during the development of an infinitely large curvature at its section (Fig. 1).

Plastic collapse is that condition of a structure for which deflections increase without change in the magnitude of a prescribed external loading.

A *collapse mechanism* is a linkage of portions of a structure for which increases in deflections may occur without change in the magnitude of a prescribed external load.

A *statically admissible distribution of bending moments* within a structure satisfies all the requirements for static equilibrium for a prescribed set of loads.

A *safe distribution of bending moments* does not exceed the plastic resisting moment anywhere in the structure.

Theorems

Static Theorem—The prescribed external loading will be equal to or less than the collapse loading when the distribution of bending moments throughout the structure is both safe and statically admissible. This also has been named the *Lower Bound Theorem.*

Kinematic Theorem—The prescribed external loading will be equal to or greater than the collapse load if calculated on the basis of an assumed collapse mechanism. This also has been named the *Upper Bound Theorem.* It follows that for all possible mechanisms, the lowest magnitude of the prescribed loading will be the collapse load.

Uniqueness Theorem—A prescribed loading will be equal to the collapse load for any safe and statically admissible distribution of bending moments in which the plastic resisting moment is developed at enough cross sections to cause collapse of the structure as a mechanism. This theorem is obtained by combining the static and kinematic theorems.

Proofs for the above theorems are available in current technical literature.[1]

Adaptation to reinforced concrete

If the moment-curvature relationship is that illustrated in Fig. 2 for reinforced concrete, there is no truly plastic resisting moment as defined above,

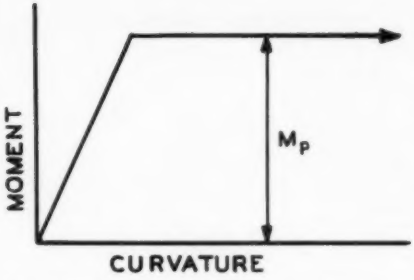

Fig. 1—Resisting moment versus curvature. Plastic resisting moment = M_p

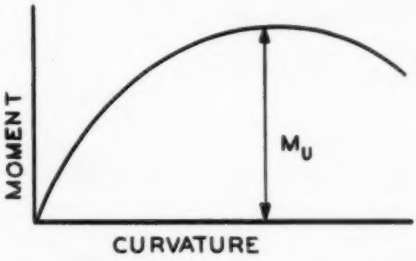

Fig. 2—Resisting moment versus curvature for reinforced concrete. No truly plastic resisting moment

nor can there be a true condition of plastic collapse. Nevertheless, it has been demonstrated experimentally[2,3,5] that at the ultimate strength values (M_u) of ACI 318-56 sufficient plastic curvature will have developed to produce a redistribution of moments in continuous beams and single-bay one-story rigid frames comparable to that developed from the moment-curvature relationship of Fig. 1, at least for steel ratios of less than about 0.03. Therefore, the Uniqueness Theorem may be restated for reinforced concrete design as follows, assuming that a "safe" distribution of bending moments does not exceed the ultimate moment anywhere in the structure:

> A prescribed loading will be equal to the ultimate load for any safe and statically admissible distribution of bending moments in which the ultimate moment is developed at enough sections to convert the structure into a collapse mechanism.

Inasmuch as the steel in reinforced concrete can be varied to fit as closely as practicable any statically admissible distribution of bending moments, it is possible to design quite simply by use of the Uniqueness Theorem. The recommendation in ACI 318-56 to calculate moments by elastic theory and then design the sections at ultimate strength for these moments corresponds to the theorem with the restriction that the distribution of moments must be elastically determined from an assumption of dimensions or relative stiffnesses of members, but such a restriction is not essential. A statically admissible distribution of bending moments is the only basic requirement for design by this theorem, and various methods of determining such distributions are presented and briefly illustrated below.

Statically admissible bending moment distributions

Funicular Polygon Method—The force and funicular polygons for a prescribed set of loads circumscribe the limits for static equilibrium and the selection of the closing ray(s) within these limits will define the conditions of support. Because the intercept between the rays at any point of a funicular polygon is proportional to the sum of the moments of all the forces to the left (or right) of the point, such a polygon will represent a statically admissible

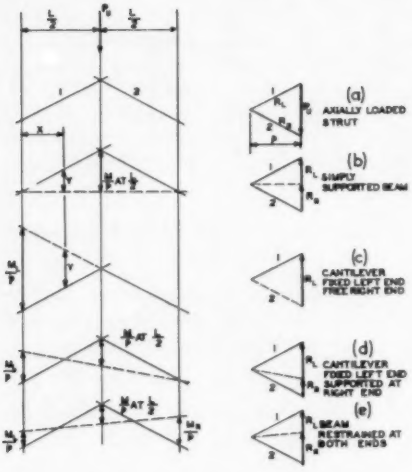

Fig. 3—Funicular polygon method

distribution of bending moments to some scale. Various applications to a beam of span length L are illustrated on Fig. 3. For example:

In Fig. 3(b):

$$\frac{Y}{X} = \frac{R_L}{p}$$

$$\therefore Yp = R_L X = M_X$$

p = pole distance of force polygon

In Fig. 3(c):

$$\frac{Y}{\frac{L}{2} - X} = \frac{R_L}{p} = -\frac{P_u}{p}$$

$$\therefore Yp = -P_u\left(\frac{L}{2} - X\right) = M_X$$

Equilibrium Equation Method—The static equations of equilibrium may be used to express relationships between the prescribed loads and critical moments, so a statically admissible distribution of internal moments is attained. Referring to Fig. 4, applications of equilibrium equations are demonstrated for the same cases (b) to (e), inclusive, of Fig. 3. The equations relating the loads and moments are:

In Fig. 4(b):

$$\Sigma M_V = 0 \quad \frac{P_u}{2} \times \frac{L}{2} - M_u = 0$$

$$P_u = \frac{4\,M_u}{L}$$

In Fig. 4(c):

$$\Sigma M_R = 0 \quad \frac{P_u\,L}{2} - M_u'' = 0$$

$$P_u = \frac{2\,M_u''}{L}$$

In Fig. 4(d):

$$\Sigma M_{R_L} = 0 \quad R_R\,L - \frac{P_u\,L}{2} + M_u'' = 0$$

$$\Sigma M_V = 0 \quad \frac{R_R\,L}{2} - M_u = 0$$

Eliminating R_R:

$$2\,M_u - \frac{P_u\,L}{2} + M_u'' = 0$$

$$P_u = \frac{2}{L}\,(2\,M_u + M_u'')$$

$$P_u = 2\,\frac{M_u}{L}\left(2 + \frac{M_u''}{M_u}\right)$$

In Fig. 4(e):

$$\Sigma M_{R_L} = 0 \quad \frac{P_u\,L}{2} + M_u' - M_u'' - R_R L = 0$$

$$\Sigma M_V = 0 \quad \frac{R_R\,L}{2} - M_u' - M_u = 0$$

Eliminating R_R:

$$\frac{P_u\,L}{2} + M_u' - M_u'' - 2\,(M_u' + M_u) = 0$$

$$P_u = 2\left(\frac{M_u' + M_u'' + 2\,M_u}{L}\right)$$

$$P_u = \frac{2\,M_u}{L}\left(\frac{M_u'}{M_u} + \frac{M_u''}{M_u} + 2\right)$$

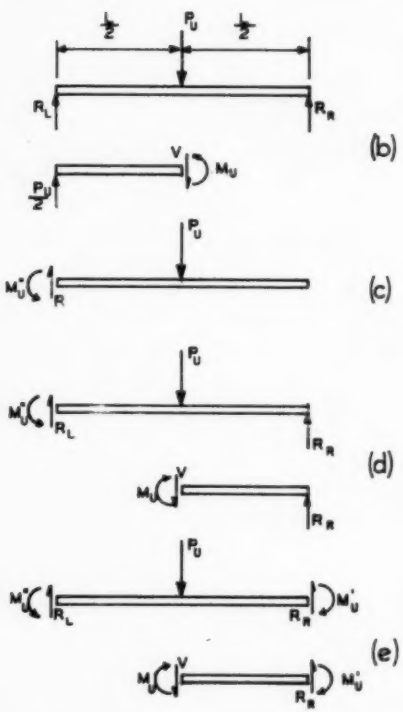

Fig. 4—Equilibrium equation method

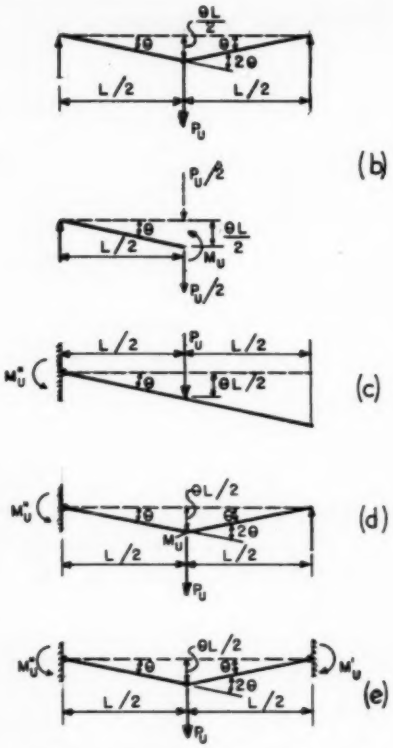

Fig. 5—Virtual displacement method

Virtual Displacement Method—The principle of virtual displacements generally is a more convenient method for obtaining the same relationships expressed by the equilibrium equation method. The principle of virtual displacements states that

> If a force system in equilibrium is subjected to an extremely small virtual displacement, consistent with its constraints, the total work done by the force system will be zero.

Cases (b) to (e), inclusive, from Fig. 3 are solved with Fig. 5 by this principle.

In Fig. 5(b):

$$\textit{External work} = 2 \times \frac{P_u}{2} \Theta \frac{L}{2}$$

$$W_e = \frac{P_u \Theta L}{2}$$

$$\textit{Internal work} = -2 \times M_u \theta$$

$$W_i = -2 M_u \theta$$

$$W_e + W_i = 0$$

$$\frac{P_u \theta L}{2} - 2 M_u \theta = 0$$

$$P_u = \frac{4 M_u}{L}$$

In Fig. 5(c):

$$W_e + W_i = 0$$

$$P_u \theta \frac{L}{2} - M'' \theta = 0$$

$$P_u = \frac{2 M''}{L}$$

In Fig. 5(d):

$$P_u \theta \frac{L}{2} - M_u'' \theta - M_u 2 \theta = 0$$

$$P_u = \frac{2 (M_u'' + 2 M_u)}{L}$$

$$P_u = \frac{2 M_u}{L}\left(\frac{M_u''}{M_u} + 2\right)$$

In Fig. 5(e):

$$P_u \theta \frac{L}{2} - M_u'' \theta - M_u 2 \theta - M_u' \theta = 0$$

$$P_u = \frac{2}{L} (M_u'' + 2 M_u + M_u')$$

$$P_u = \frac{2 M_u}{L}\left(\frac{M_u''}{M_u} + 2 + \frac{M_u'}{M_u}\right)$$

It may be noted that only the internal moments at the critical sections and the external loads do work for a properly selected virtual displacement. Therefore, it is not necessary to sketch the component parts of the structure separately, but only to show the mechanism being considered.

Superposition—Statically admissible moment diagrams may be superposed on one another for their corresponding load and reaction combinations. If the ultimate moment is developed (but not exceeded anywhere) at enough sections by the resulting statically admissible diagram to convert the structure into a collapse mechanism, the combined loadings will be the collapse load. Furthermore, for design the steel content may be varied such that the ultimate moment is developed at enough sections to produce a collapse mechanism for any statically admissible moment diagram obtained by superposition, and the combined loadings will be the collapse load.

Elastic Compatibility—It is significant that for any statically admissible distribution of bending moments there will exist a structure for which an analysis by the theory of elastic frames would yield that particular distribution of moments. For such cases, elastic compatibility would be retained essentially up to the ultimate load.

Simple rigid frames

Although the equilibrium equation and funicular polygon methods are also applicable to rigid frames, it is more convenient to use the virtual displacement method on a collapse mechanism analyzed as a kinematic linkage of structural parts.

Two examples are illustrated in Fig. 6, from which the load-moment equations listed below have been obtained by applying the principle of virtual displacements. The use of the instantaneous center of rotation (see any elemen-

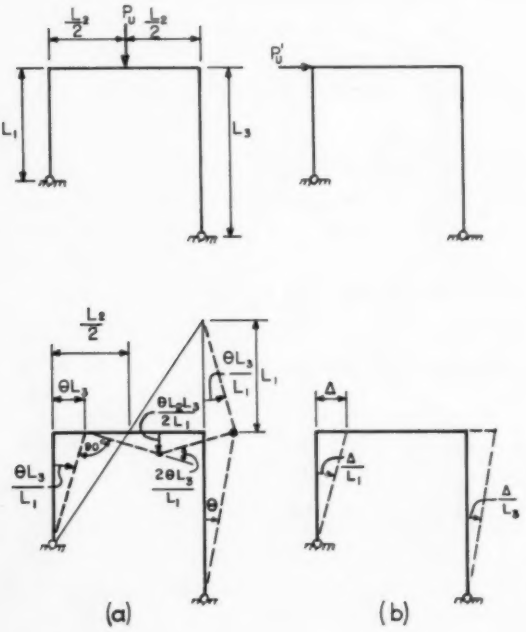

Fig. 6—Mechanisms for simple frames

tary kinematics text) facilitates the solutions, but is not essential. Moment diagrams for the various members may be constructed readily by the customary procedures. The right-hand knee moment is M_u', and the positive moment at $L/2$ is M_u. Span lengths are defined in the figure.

In Fig. 6(a):

$$P_u = \frac{4\,M_u}{L_2} + \frac{2\,M_u'}{L_2}\left(1 + \frac{L_1}{L_2}\right)$$

$$= \frac{2\,M_u}{L_2}\left[2 + \frac{M_u'}{M_u}\left(1 + \frac{L_1}{L_2}\right)\right]$$

In Fig. 6(b):

$$P_u' = \frac{M_u'}{L_1}\left(\frac{M_u''}{M_u'} + \frac{L_1}{L_2}\right)$$

M_u'' = left-hand knee moment

CONCENTRATED PLASTIC ROTATIONS

The magnitude of the concentrated plastic rotation (Θ_p) necessary for the attainment of M_u at all critical sections may be calculated from a consideration of the lack of balance or closure of the M_u/E_cI diagram[4] or by any other equivalent modification of elastic theory to include plastic rotations. Examples for continuous beams and simple rigid frames are given below. It is also true that M_y or M_c, the yield moment or the crushing moment, may be used in place of M_u if residual plastic rotation is desired in the design for a specific load requirement.[3]

Continuous beams

If the load P on the two-span continuous beam of Fig. 7a is just large enough to develop M_u at the first critical section, the moment diagram will be that shown in Fig. 7b. The accumulated plastic angle change of the solid black area of Fig. 7c will slightly increase the negative moment above that for purely elastic conditions, by means of a small amount of plastic adjustment. Although up to this load the dominant action of the beam has been elastic, further load

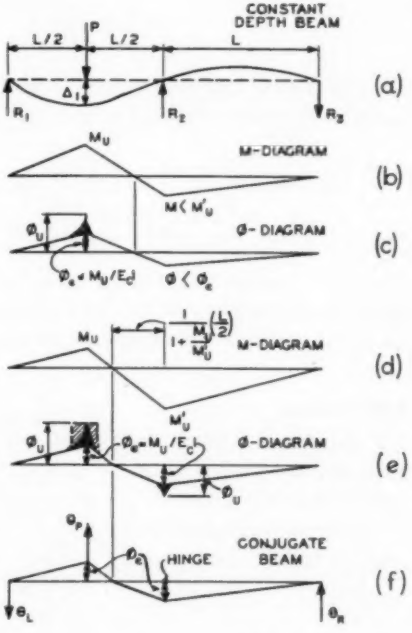

Fig. 7—Development of Θ_p in two-span continuous beam with load at center of one span only

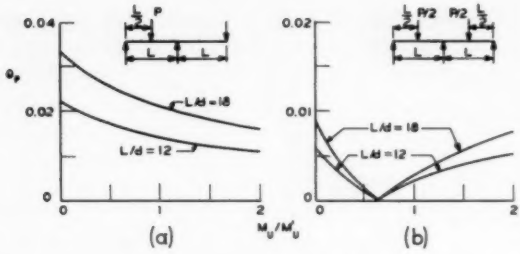

Fig. 8—Calculated Θ_p requirements for two-span continuous beams

increases will cause spread of ϕ_u in the vicinity of the load along with a flexural adjustment until the ultimate moment develops at the center support as shown in Fig. 7d.

The accumulated plastic angle change in the vicinity of the load in Fig. 7e is represented by the concentrated plastic rotation Θ_p on the idealized conjugate beam of Fig. 7f, in which the small amount of plastic rotation at R_2 has been considered negligible. This conjugate beam may be used to obtain the magnitude of Θ_p necessary for the formation of the moment diagram of Fig. 7d. The Θ_p equation for this case is:

$$\Theta_p = \frac{\phi_c L}{12\left(1 + \frac{M_u}{M_u'}\right)^2}\left[2\left(\frac{M_u}{M_u'}\right)^2 + 15\frac{M_u}{M_u'} + 11\right]$$

Similar solutions provide the following equation for the two-span continuous beam with concentrated loads at the center of each span.

$$\Theta_p = \pm \frac{\phi_c L}{12\left(1 + \frac{M_u}{M_u'}\right)^2}\left[6\left(\frac{M_u}{M_u'}\right)^2 + \frac{M_u}{M_u'} - 3\right] \qquad \begin{cases} + \text{ if } \Theta_p \text{ occurs at } R_2 \\ - \text{ if } \Theta_p \text{ occurs at } L/2 \end{cases}$$

Fig. 8 illustrates the variation of Θ_p with different ratios of critical moments for these two cases. It should be noted that for the symmetrical loading condition an optimum ratio of critical moments exists for which the value of Θ_p is zero.

Rigid frames

Following a similar method of analysis for the rigid frames of Fig. 6, the Θ_p equations are:

In Fig. 6(a):

$$\Theta_p = \frac{\phi_c L_2}{12}\left(8\frac{L_3}{L_2} + a^2 - 3a + \frac{L_1}{L_3}\left[a^2 - 2a + 1\right] - 2b^2 - 7b + 5 \right.$$

$$\left. \frac{-2\,L_1/L_3}{1 + \frac{L_1}{L_3}}\left\{a^2 + \frac{L_1}{L_3}[a^2 + a - 2] - 2b^2 - b + 2 - \frac{4}{L_3 L_2'}\left[L_1^2 - L_3^2\right]\right\}\right)$$

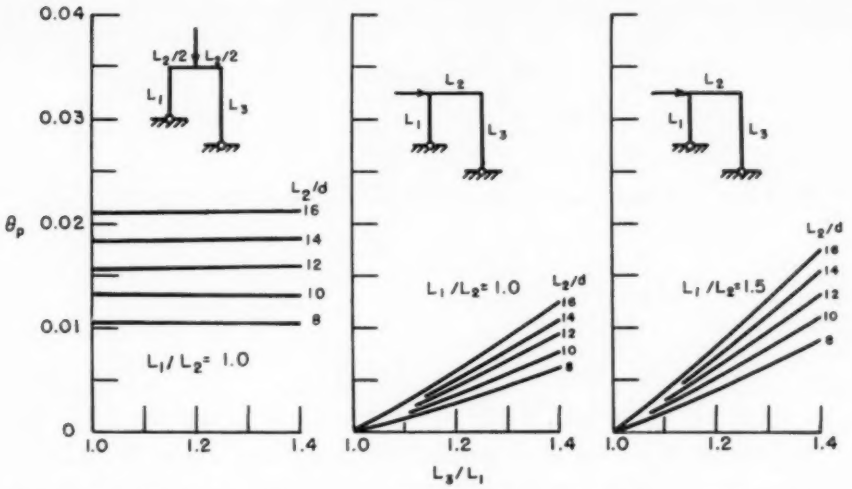

Fig. 9—Calculated Θ_p requirements for frames, no support deflections, with side-sway

$$\text{where } a = \frac{M_u/M_u'}{(L_1/L_2) + (M_u/M_u')}, \quad b = \frac{M_u/M_u'}{1 + (M_u/M_u')}, \quad \text{and } \phi_c = \frac{M_u}{E_c I}$$

In Fig. 6(b):

$$\Theta_p = \frac{\phi_c L_2}{6}\left(2[a']^2 - 5a' + 1 + \frac{L_3}{L_1}\left\{-2[a']^2 - a' + 2 + 2\frac{L_2}{L_2}\right\} - 2\frac{L_1}{L_2}\right)$$

$$\text{where } a' = \frac{M_u/M_u'}{\dfrac{M_u}{M_u'} + 1}$$

For such frames with a constant value of M_u and d, the relationship between Θ_p and the frame proportions will be as shown in Fig. 9.

DEFLECTIONS

Deflections at ultimate load may be obtained by using the same conjugate beam procedure as presented above for obtaining Θ_p, by determining the slope at either end of the two-span beam and then the deflection at the desired point as the internal moment in the conjugate beam. Expressions for the various cases previously discussed are provided below for deflections at the load.

Continuous beams

The midspan ultimate deflection equations for the continuous beams previously discussed are as follows:

In Fig. 7:

$$\Delta_1 = \frac{\phi_e L^2}{24\left(1 + \frac{M_u}{M_u'}\right)^2}\left(3\left[\frac{M_u}{M_u'}\right]^2 + 11\frac{M_u}{M_u'} + 6\right)$$

With concentrated loads at the center of each span:

$$\Delta_1 = \begin{cases} \dfrac{\phi_e L^2}{48\left(1 + \frac{M_u}{M_u'}\right)^2}\left(4\left[\frac{M_u}{M_u'}\right]^2 + 7\frac{M_u}{M_u'} + 1\right) & \text{if } \Theta_p \text{ occurs at } R_2 \\ \dfrac{\phi_e L^2}{24\left(1 + \frac{M_u}{M_u'}\right)^2}\left(2 + 3\frac{M_u}{M_u'} - \left[\frac{M_u}{M_u'}\right]^2\right) & \text{if } \Theta_p \text{ occurs at } L/2 \end{cases}$$

Fig. 10 and 11 provide a comparison of the deflection history of these two cases as the loading is increased from zero to the collapse load.[6] It is evident that each case will have elastic deflections at working loads for the ratios of M_u/M_u' shown.

Rigid frames

The midspan and side-sway ultimate deflection equations for the rigid frame with concentrated vertical load are:

Midspan:

$$\Delta_1 = \frac{\phi_e L_2^2}{24}\left(4\frac{L_3}{L_2} - 2b^2 - b + 2 - \frac{L_1}{L_1 + L_2}\left\{a^2 + \frac{L_1}{L_3}[a^2 + a - 2] - 2b^2 - b + 2 - \frac{4}{L_3 L_2}[L_1^2 - L_3^2]\right\}\right)$$

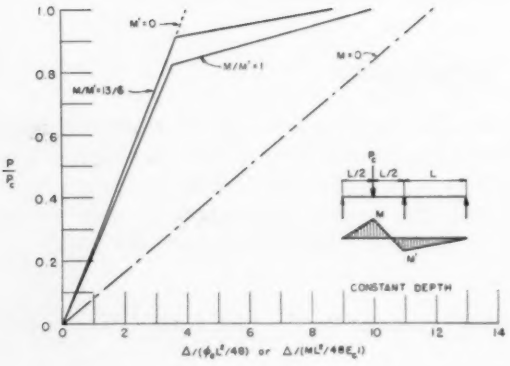

Fig. 10—Deflection history for loading as it is increased from zero to collapse load for Θ_p at R_2

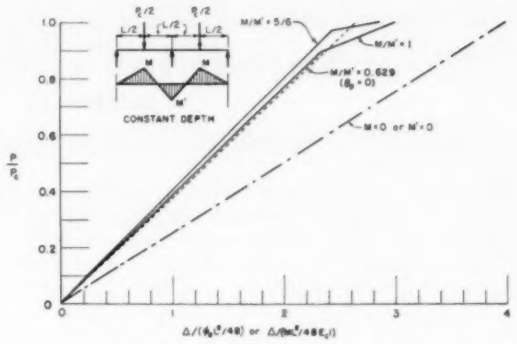

Fig. 11—Deflection history for loading as it is increased from zero to collapse load for θ_p at $L/2$

Side-sway:

$$\Delta = \frac{L_1 L_3}{L_1 + L_3}\left(\frac{\phi_e L_2}{12}\right)\left(a^2 + \frac{L_1}{L_2}[a^2 + a - 2] - 2b^2 - b + 2 - \frac{4}{L_3 L_2}[L_1^2 - L_2^2]\right)$$

$$\text{where:} \quad a = \frac{M_u/M_u'}{(L_1/L_3) + (M_u/M_u')}$$

$$b = \frac{M_u/M_u'}{1 + (M_u/M_u')}$$

The side-sway ultimate deflection equation for the rigid frame with the horizontal load is:

$$\Delta = \frac{\phi_e L_2 L_3}{6}\left(2\frac{L_3}{L_2} - 2[a']^2 - a' + 2\right)$$

$$\text{where } a' = \frac{M_u/M_u'}{\dfrac{M_u}{M_u'} + 1}$$

SUMMARY

1. Limit design is adaptable to any statically admissible distribution of moments, if the required concentrated plastic angle change(s) can be developed by the material used.

2. The required plastic angle changes for the two-span beams and one-story frames in Fig. 8 and 9 do not exceed the capability of reinforced concrete sections shown by recent tests.[2,3]

3. Deflections at working loads for the cases studied are well within the elastic range for all ratios of positive to negative critical moments studied.

4. Deflections upon the attainment of the ultimate strength at all critical sections compare favorably for the statically indeterminate cases studied.

5. It appears that limit design may be a powerful additional tool for the designer of continuous beams and frames of reinforced concrete.

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Title No. 56-20

Problems and Performance of Precast Concrete Wall Panels

By VICTOR F. LEABU

The use of prefabricated concrete wall panels for curtain walls is gaining popularity in the building industry. However, the precast concrete wall panel in service can cause problems. While most walls of this type are trouble-free, defects in some precast walls are forcing those concerned to take a closer look at design and construction practices.

Since concrete is the basic material in the panels, the inherent problems for concrete must be controlled. Variation in color, inefficient *U* factor, and bulging are problems of precast wall panels inherited from concrete. Corrective measures for some of these problems are presented.

■ THE PRECAST CONCRETE WALL PANEL, the concrete industry's newest product, is gaining in popularity in the building industry. The many advantages of its novel construction, such as light weight, speed of erection, resistance to wind, rain, and fire, and low initial cost with minimum maintenance, have sparked the enthusiasm of architects, engineers, builders, and the public.

The thin metal curtain wall with its problems of leakage, condensation, heat transmission, and warping, or the masonry wall with its problems of cracks, shrinkage control, and excessive weight should suggest to the architect and engineer that the precast concrete curtain wall has its own problems that only proper education and communication of ideas, experience, and technical data among those concerned will overcome. Concrete expands, swells, contracts, and shrinks with variations in moisture, temperature and weathering cycles. To expect the precast concrete wall panel to be free of these difficulties would be asking too much.

DESIGN EVOLUTION

The evolution in precast wall panels can be seen in the experience at our office which has used them in industrial, public, and commercial buildings over the past 9 years. The first panels were solid concrete, 8 in. thick, averaging 6 ft 6 in. wide x 20 ft long and used only as sill walls. These panels were fabricated on the site and tilted into position. They gradually changed in size and thickness and are now usually sandwich type and only 5 in. thick with increased length and width. Panels which were normally used as sill walls supported on grade beams are now suspended between columns and span

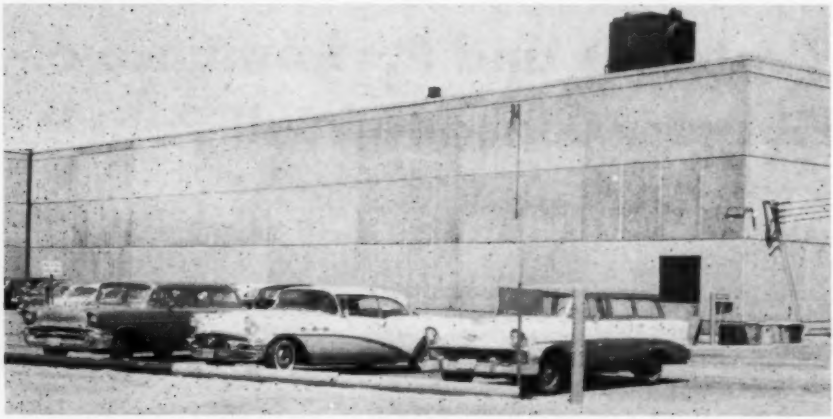

Fig. 1—Precast sandwich panels with broomed finish made with gray cement. After 3 years' exposure, uneven color tone is still noticeable

structurally from column to column. The first panels had a smooth, finished surface; today's surface treatments are more attractive, such as exposed aggregates, mosaic, glass and marble chips, quartz aggregates with white sand and cement, or other decorative exterior finishes and textures.

The evolution did not occur without introducing problems, aside from those encountered in fabrication and erection well known to the industry. Generally the most troublesome problems encountered with the precast panels can be summarized as:

1. Variations in color or discoloration of panels
2. Maintaining a design factor of heat transmission (U factor)
3. Bulging of panels

VARIATION OF COLOR AND DISCOLORATION

The ordinary precast concrete wall panel is seldom a work of art. Its monotonous color tone and shades of gray are a constant source of criticism. Until recently, most buildings of this type have been far from attractive. This method of construction and mechanization has only served as an inexpensive means of durable construction and fast erection.

The precast wall shown in Fig. 1 is a good example of troweled or broomed textured panels using regular concrete. These panels were erected approximately 3 years ago. Time and weathering have not yet harmonized them to an even tone or color shade.

To some architects and clients this inherent appearance is acceptable, however, this kind of color variation is not always accepted without criticism. To obtain uniformity in color and texture requires even more control of concrete and its ingredients than is normally practical. Accelerated curing and fabrication of panels, a common practice in the industry, makes inspection

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and matching of panels even more difficult since aging and exposure will change the shade of panels for some time after erection.

To minimize shade variation and discoloration of panels and to make the precast panel more attractive the following measures are suggested:

1. Cement and aggregates should be from the same sources throughout the job. When using gray cement periodic checks should be made at the supplier for variation in shades of gray. No two suppliers have the same shade of gray cement.
2. In erection, some attempt should be made to match panels to minimize shade variations. Variation of color should be kept to a minimum, especially in adjacent panels.
3. The client and architect should be properly educated as to what to expect in the regular concrete panel and be prepared to accept the end results.
4. White cement should be used where shades of gray are not acceptable, keeping in mind that even white cement has a blue or yellow cast, depending on the manufacturer.
5. Textured panels will reduce the monotony of a smooth, troweled surface.
6. Methods of curing and storage of panels before erection should be uniform for all panels.
7. Surface treatments, such as exposed aggregate, mosaic, or marble chips, quartz aggregates with white sand and cement, or other decorative exterior finishes, will overcome the monotony of a plain concrete panel, but with additional cost.
8. A sharp contrast in panel shade by the use of colored cements may be used effectively in some pattern to relieve monotony of natural appearance of regular concrete provided that the colored cements retain their basic color without fading.
9. Avoid the use of exposed steel or iron inserts, edge angles, etc., that will rust and stain finished surfaces.

HEAT TRANSMISSION

The U factor of any wall or roof material is important in the design of buildings. For those not familiar with the significance of this mechanical engineer's term, it tells how much heat, in Btu's passes through a square foot of any given material in 1 hr for each degree difference in temperature (Fahrenheit), between the exterior and interior faces.

Too often, responsibility for the determination of the thermal properties of wall and roof materials is divided between the architect and mechanical engineer. This divided responsibility usually leads to confusion and neglect. However, on this divided responsibility depends to a large degree the lifetime comfort levels within a building, the initial cost of heating and cooling equipment, as well as their annual operating and maintenance cost.

With precast concrete wall panels, U varies considerably with the type of panel and the type of concrete used. The greatest heat loss occurs through solid regular concrete panels with less heat loss through solid lightweight,

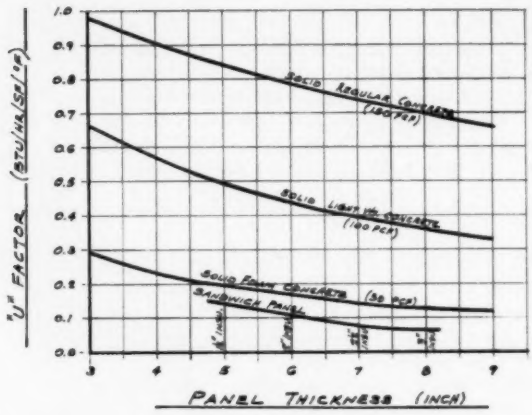


Fig. 2—*U* factor for various thicknesses of solid panels made with regular, lightweight, and foamed concrete, and sandwich panels

cellular, or foamed concrete. The most efficient precast concrete panel is the sandwich type made with regular or lightweight concrete with a layer of insulation such as foamglass, Styrafoam, or Fiberglas sandwiched between two layers of concrete. Fig. 2 gives the relative *U* factors of the different types of solid regular, lightweight, and foamed concrete panels together with sandwich panels for several panel thicknesses.

Insulated panels are usually designed with a *U* ranging from 0.15 to 0.20. This is usually computed on the cross section through the insulation. The

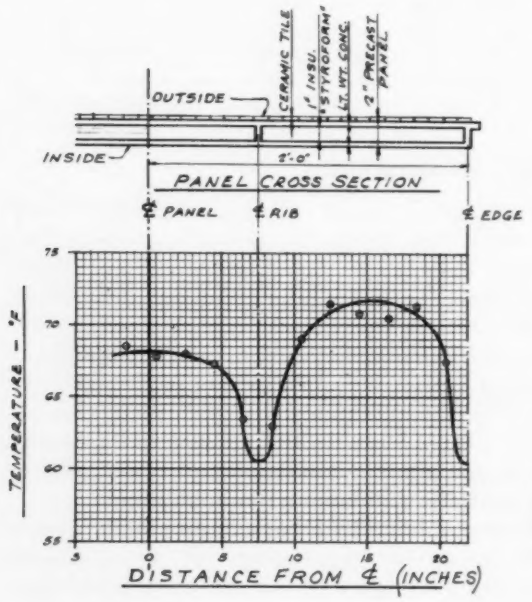
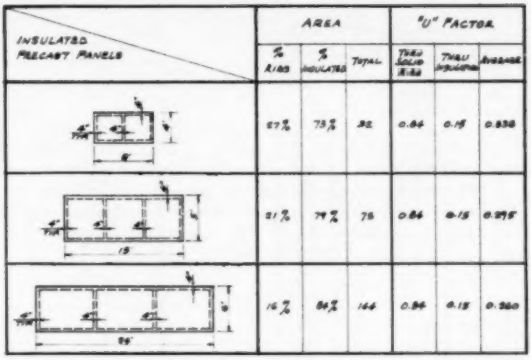


Fig. 3—Temperature variations through solid ribs and stiffeners in relation to insulated portion of sandwich panels

Fig. 4 — Typical sandwich panels designed for a U factor of 0.19. Actual factors as affected by ribs



effect of ribs and solid sections in the panel is seldom considered. This effect, however, is great enough to increase the U factor of the panel as much as 50 percent.

Fig. 3 shows temperature variations through solid ribs and stiffeners in relation to the insulated portion of precast sandwich panels. It is evident that ribs and solid concrete sections of a sandwich panel using a highly efficient insulation core reduces its insulation qualities considerably, not only at the ribs, but also for some distance beyond the solid concrete sections. To illustrate this effect, some typical precast concrete sandwich panels designed for a U of 0.19 are shown in Fig. 4. This factor was represented by the precast industry as calculated through the insulated portion only. The actual value for the over-all panel (including ribs, etc.) shows a factor from 0.26 to 0.33. This represents an increase of over 40 percent. The ratio of rib area to the insulated area of panel has a decided effect on the U factor, a high percentage of rib area will increase it considerably. Where possible, ribs and solid concrete stiffeners should be kept to a minimum or eliminated if structural design conditions permit. Joints and intersections between panels should be studied and designed to maintain a factor comparable to the rest of the precast concrete panel.

BULGING

This phenomenon is characteristic in many precast concrete panels and more pronounced in the sandwich type than in the solid type.

Concrete being the basic material of precast panels, a constant state of movement can be expected due to changes in temperature and atmospheric conditions.

Bulging and warping of precast panels can result in unsightly cracks at ceiling, floor, and partition lines unless proper provision is made in the design of panel and connecting components. Bulging may be caused by temperature, moisture, and curing and shrinkage differences between interior and exterior faces.

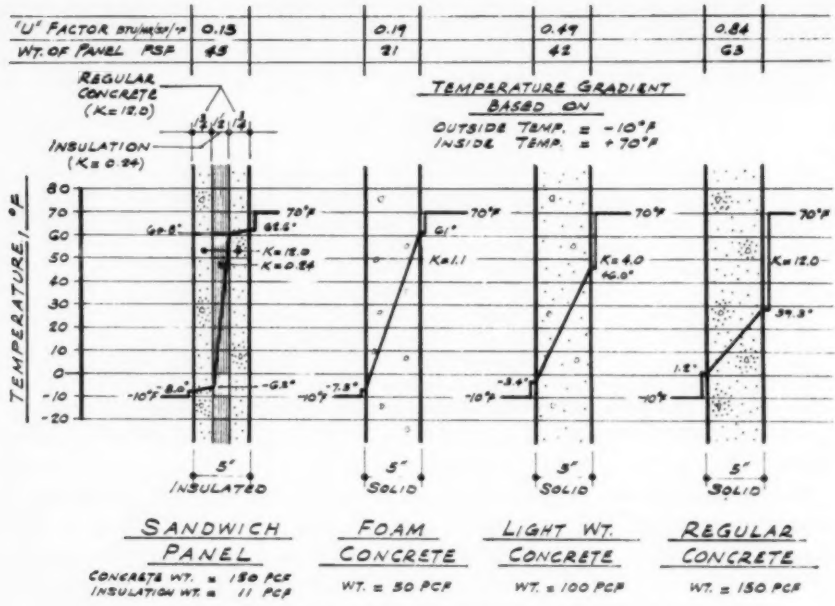


Fig. 5—Temperature gradient through four types of precast wall panel

Temperature effects

Temperature differential appears to have the greatest effect on the precast concrete wall panels and especially on the sandwich type. Fig. 5 shows the temperature gradient through four different types of precast concrete panels for a temperature range of −10 F outside to 70 F inside. Note that the sandwich panel with its highly efficient insulation has the greatest temperature differential and the solid regular concrete panel the least. In the sandwich panel a temperature variation of approximately 68 F is possible between the exterior and interior layers of concrete.

Computation of temperature effects

The horizontal displacement due to temperature differential can be computed for regular (150 lb per cu ft) and lightweight (100 lb per cu ft) concrete as follows.

Curvature is computed for two support conditions (See Fig. 6 for typical details):

1. Panel supported as a beam on pin connections with equivalent moment at ends induced by the length change due to temperature differential. This represents a panel spanning between columns and supported at ends only.

2. Panel acting as a beam fixed at supports with equivalent moment at ends induced by length change due to temperature difference. This represents panels with rigid support between panel ends such as ties to floor and roof spandrels.

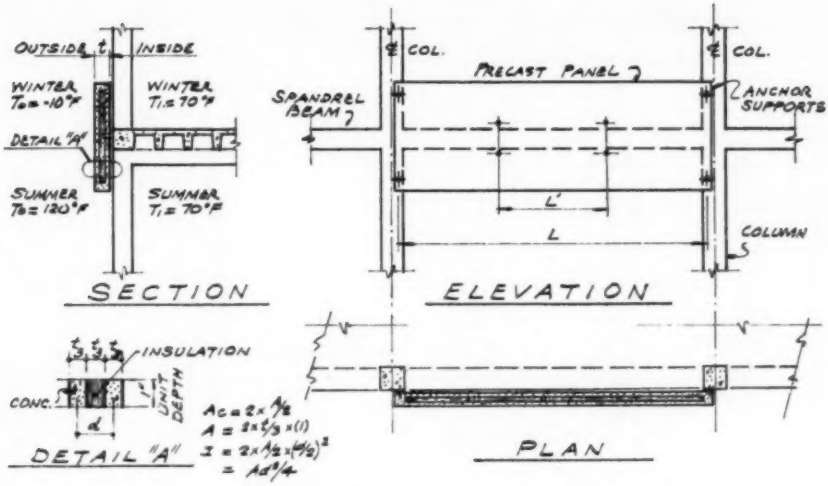


Fig. 6—Typical panel support and details

Notation

T_o = Temperature at outside, deg F
T_i = Temperature at inside, deg F
δT = Difference of temperature between outside and inside, deg F
c = Coefficient of expansion, in. per in. per deg F
Regular concrete, $c = 0.0000079$ per deg F ($K = 12.0$)
Lightweight concrete, $c = 0.0000042$ per deg F ($K = 4.0$)
L = Length of panel (or length between supports depending on the restraint of connections), in.
d = Effective depth of panel, distance between center of gravity of outside and inside faces, in.
e = Elongation or shortening, in.
Δ = Horizontal displacement, in.
A = Area of concrete, sq in.
I = Moment of inertia
E = Modulus of elasticity of concrete, psi
K = Thermal conductivity, Btu per hr per sq ft per deg F per in. thickness

Assume no change in length at neutral axis.
Exterior face elongated = + e
Interior face shortened = − e

Support Condition 1

$$\pm e = c \times \frac{\delta T}{2} \times L$$

$$\pm e = \frac{PL}{E} \times \frac{1}{A/2}$$

$M = Pe$
$P = \frac{1}{4}c \times \delta T \times AE$
$M = \frac{1}{4}ce \times \delta T \times AE$

$$I = \frac{Ad^2}{4}$$

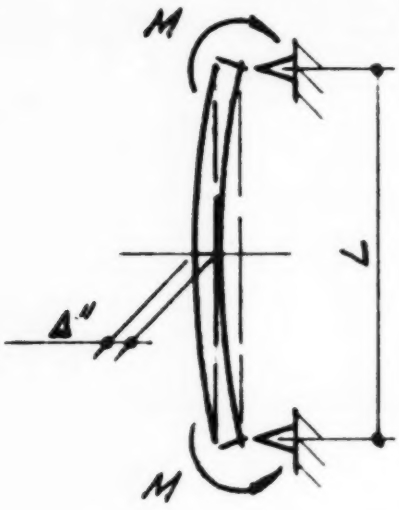
Support Condition 2

$$\pm e = c \times \frac{\delta T}{2} \times l$$

$$\pm e = \frac{Pl}{E} \times \frac{1}{A/2}$$

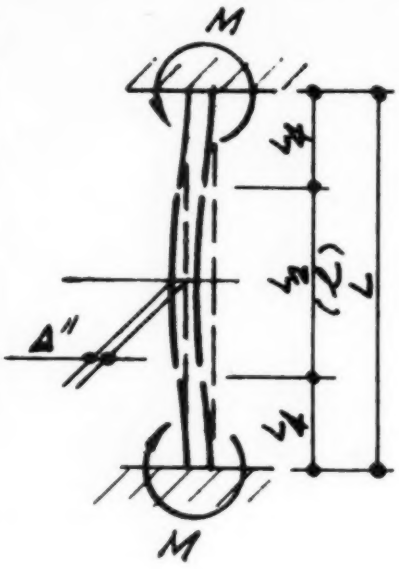
$M = Pe$
$P = \frac{1}{4}c \times \delta T \times AE$
$M = \frac{1}{4}ce \times \delta T \times AE$

$$I = \frac{Ad^2}{4}$$



$$\Delta = \frac{ML^2}{8EI} = \frac{cd \times \delta T \times AL^2}{32I}$$

$$\Delta = \frac{c \times \delta T \times L^2}{8d}$$



$$\Delta = \frac{Ml^2}{8EI} = \frac{ML^2}{32EI} = \frac{cd \times \delta T \times AL^2}{128I}$$

$$\Delta = \frac{c \times \delta T \times L^2}{32d}$$

Table 1 shows deflections of regular and lightweight concrete for both support conditions, and panel lengths from 1-30 ft for 1 in. of effective depth and 10 deg F temperature differential. Lateral deflection versus span length for a typical 5-in. sandwich panel for an 80 F temperature differential is shown in Fig. 7. These curves show the effect of panel support and type of concrete on bulging. From the curves it is evident that the use of lightweight concrete and continuity of panels at points of support reduced the curvature of panels due to temperature differential. Of course in actual practice, panel support is seldom fully fixed or simply supported but more likely between these two extremes.

Field observation of lateral panel movement due to temperature seems to correlate the computed deflections shown in Fig 7. However, no research on panel movement due to temperature on this type of wall has been performed, to the writer's knowledge.

In some field observations, the effect of temperature differential cannot always explain the outward movement of the panels. Panels seldom curve inward, and in most field observations the curvature is outward regardless of whether the hot face of the panel is on the outside or inside of the building. In most installations, the building frame would restrict inward movement but this is probably not the only reason.

TABLE 1—PANEL DEFLECTIONS FOR 10 DEG F TEMPERATURE DIFFERENTIAL PER INCH OF EFFECTIVE PANEL DEPTH

PANEL LENGTH L' (FT.)	DEFLECTION (INCHES) Δ = INCH FOR ΔT=10° d = 1" (L)²	REGULAR CONCRETE C = 0.0000079 IN./IN./DEG F Δ=0.001422 L²	REGULAR CONCRETE C = 0.0000079 IN./IN./DEG F Δ=0.000356 L²	LIGHT WT. CONCRETE C = 0.0000042 IN./IN./DEG F Δ=0.000755 L²	LIGHT WT. CONCRETE C = 0.0000042 IN./IN./DEG F Δ=0.000189 L²
1'	1	0.001	0.000	0.001	0.000
2'	4	0.006	0.001	0.003	0.001
3'	9	0.013	0.003	0.007	0.002
4'	16	0.023	0.006	0.012	0.003
5'	25	0.036	0.009	0.019	0.005
6'	36	0.051	0.013	0.027	0.007
7'	49	0.070	0.018	0.037	0.009
8'	64	0.091	0.023	0.048	0.012
9'	81	0.115	0.029	0.061	0.015
10'	100	0.142	0.036	0.076	0.019
11'	121	0.172	0.043	0.092	0.023
12'	144	0.205	0.051	0.109	0.027
13'	169	0.240	0.060	0.128	0.032
14'	196	0.279	0.070	0.148	0.037
15'	225	0.314	0.080	0.170	0.043
16'	256	0.364	0.091	0.194	0.048
17'	289	0.411	0.103	0.218	0.055
18'	324	0.461	0.115	0.245	0.061
19'	361	0.514	0.128	0.273	0.068
20'	400	0.570	0.142	0.302	0.076
21'	441	0.627	0.157	0.333	0.083
22'	484	0.689	0.172	0.366	0.092
23'	529	0.753	0.188	0.400	0.100
24'	576	0.820	0.205	0.435	0.109
25'	625	0.890	0.222	0.473	0.118
26'	676	0.962	0.240	0.511	0.128
27'	729	1.040	0.260	0.551	0.138
28'	784	1.118	0.280	0.592	0.148
29'	841	1.200	0.300	0.635	0.159
30'	900	1.280	0.320	0.680	0.170

Moisture differential

Concrete reacts to moisture changes in the same manner as it does to temperature variations. Concrete swells with increase in moisture and shrinks when the moisture is decreased. Precast panels with one side exposed to changes in weather and moisture conditions and a relatively constant moisture condition on the other are subject to constant movement and volumetric change. The amount of distortion depends on several factors, the most important of which are:

1. Climatic conditions outside and inside of the building
2. Type of panels—solid or sandwich
3. Porosity and absorption rate of the concrete face exposed to the elements
4. Panel size

The core or filler of sandwich panels acts as a good vapor barrier, as well as insulation. For this reason a greater variation in moisture can be expected between the inside and outside face. With the inside concrete layer held at a relatively constant moisture condition, the outside concrete layer is subject to greater volumetric change due to atmospheric exposure, resulting in a curvature of the panel. Solid precast panels due to their homogeneous nature have a much more uniform moisture gradient and as a result will have less curvature due to moisture differential.

The porosity or absorption rate of the exposed concrete surfaces effects the volumetric expansion and shrinkage. To keep panel movement due to moisture to a minimum, the concrete and its ingredients should have a low absorption rate. Closed cells and watertight concrete surfaces are desirable and reduce bulging. Panel size and thickness play an important part in the magnitude of curvature that can be expected. Panels exceeding 20 ft in width or length, and less than 5 in. thick, will have excessive movement.

Curing shrinkage

Shrinkage of concrete while curing presents a problem not only during fabrication but also after erection. Warping of panels due to differential shrinkage, especially in sandwich type panels, is difficult to control. Casting of panels in a flat position simulates the approximate conditions of placing concrete slabs on the ground. The rate of curing and evaporation of moisture of the two faces are not uniform and usually result in warped surfaces. Accelerated curing, removal from forms as soon as practical, and storing panels with equal exposure on both faces until erected prevents excessive warping. Low

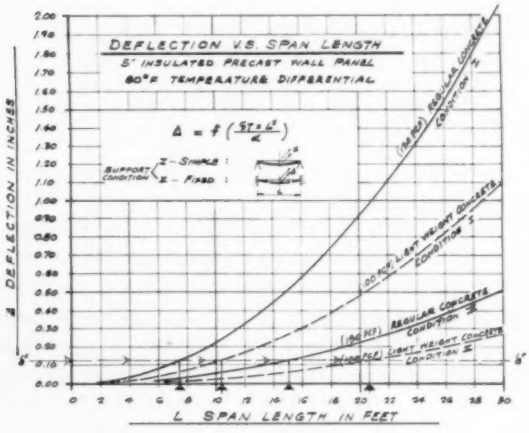

DEFLECTION VS. SPAN LENGTH
5" INSULATED PRECAST WALL PANEL
80°F TEMPERATURE DIFFERENTIAL
Δ = f (5L⁴ / d)
SUPPORT CONDITION
Σ - SIMPLE
Σ - FIXED
(150 PCF) REGULAR CONCRETE CONDITION Σ
(90 PCF) LIGHT WEIGHT CONCRETE CONDITION Σ
(150 PCF) REGULAR CONCRETE CONDITION Σ
(90 PCF) LIGHT WEIGHT CONCRETE CONDITION Σ
Δ DEFLECTION IN INCHES
L SPAN LENGTH IN FEET

Fig. 7 — Lateral deflection versus span length for a typical 5-in. sandwich panel

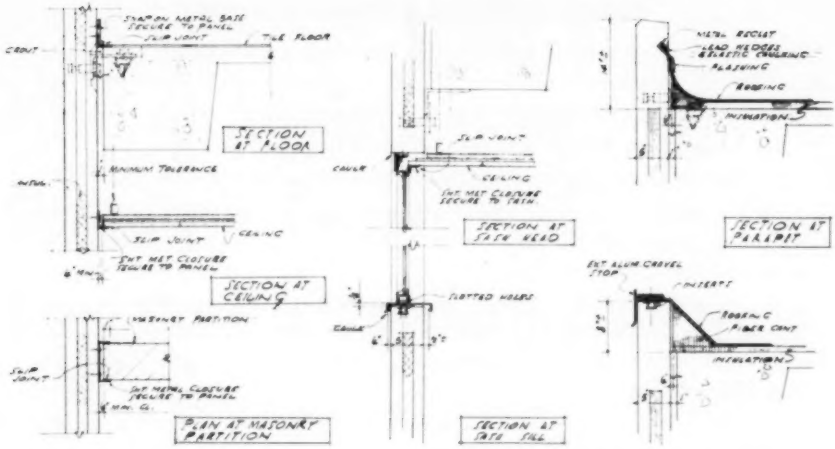


Fig. 8—Suggested details to avoid cracking and allow for panel movement

slump concrete and the use of curing compounds, water saturated covering, and steam curing processes will also help.

Since curing and measurable shrinkage of concrete can continue for as long as 3 years, movement of concrete panels can be expected long after erection due to nonuniform exposure of the two faces. This unequal shrinkage can possibly explain the observed outward instead of inward bulging of precast panels.

DESIGNING FOR LATERAL PANEL MOVEMENT

It is possible to design components of a building frame, such as ceiling, floor, roof, and partition joints intersecting the panel walls to compensate for some lateral movement. Fig. 8 shows some suggested details which will avoid unsightly cracks in architectural finishes and provide for panel movement assuming a maximum allowable lateral deflection of $\frac{1}{8}$ in.

From Fig. 7, for an allowable lateral deflection of $\frac{1}{8}$ in. due to temperature differential for a 5 in. precast sandwich panel, the following dimensions are suggested as approximate maximums:

1. Regular concrete and Support Condition 1, 8 ft
2. Regular concrete and Support Condition 2, 15 ft
3. Lightweight concrete and Support Condition 1, 11 ft
4. Lightweight concrete and Support Condition 2, 21 ft

To account for panel movement, connections to the building frame must be carefully designed. Ductility and strength are prime requisites of good connector material. Brittle materials such as cast iron should not be used if connection failure is to be avoided.

CONCLUSIONS AND RECOMMENDATIONS

Some of the problems of precast concrete wall panels can be minimized if the architect, engineer, and precast industry realize that good service performance of precast concrete wall panels is possible,

1. Through exchange of design, experience, and research
2. By developing better and more pleasing surface treatments
3. By designing heating and cooling systems for over-all panel U factors with proper allowances for the effect of solid ribs and stiffeners on heat and cooling losses
4. By designing ceiling, floor, and partition joints which intersect precast wall panels to allow a predetermined amount of lateral panel movement without developing unsightly cracks due to temperature and moisture differentials
5. By recognizing that the sandwich type precast panels are more susceptible to bulging than solid panels
6. By using smaller panels to keep lateral movement within acceptable tolerance
7. Through the use of lightweight structural concrete or foamed concrete which has a smaller coefficient of expansion than regular concrete.

ACKNOWLEDGMENTS

The writer wishes to thank Peter Hsi, structural engineer, Giffels and Rossetti, for his help in developing and preparing the charts and graphs and Giffels and Rossetti, architects and engineers, Detroit, for access to panel design data and details.

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Title No. 56-21

Effect of Admixtures on Electrolytic Corrosion of Steel Bars in Reinforced Concrete

By YASUO KONDO, AKIHIKO TAKEDA, and SETSUJI HIDESHIMA

Describes experimental research on the effects of admixtures, fly ash and $CaCl_2$ with and without calcium lignosulfonate, on the electrolytic corrosion of reinforcing steel embedded in 5⅞-in. concrete cubes. The reinforcing steel was subjected, as anode or cathode, to direct current of 5, 10, and 20 v, or to alternating current of 20 v for 28 days, 3 hr each day.

Small amounts of $CaCl_2$, less than 1 percent of cement by weight, caused no harmful reaction under direct current of less than 10 v, and when $CaCl_2$ and calcium lignosulfonate were used together, the corrosion tendency was alleviated considerably even under direct current of 20 v. Alternating current of 20 v caused no corrosion in concrete containing the admixtures.

■ IT IS A FEATURE OF ELECTRIC RAILROADS in Japan that the rails of the track are utilized as cathodes by which the current returns, and since it is both technically and economically difficult to prevent all leakage of electricity from the rails, some straying of current is inevitable. It is generally believed that electrolytic corrosion of steel reinforcing bars in concrete caused by these stray currents is accelerated when fly ash and calcium chloride are used as admixtures, and it was the purpose of this investigation to ascertain by experimental methods the amount of admixture permissible as well as to discover some way of preventing the damage. With this purpose in view, four different types of reinforced concrete were made using fly ash or calcium chloride with and without a calcium lignosulfonate admixture. These were subjected to direct currents of 0, 5, 10, and 20 v using the reinforcing bars as anodes or cathodes. Tests were also conducted for 20 v alternating currents. The aspects of corroding steel were observed through such phenomena as changes in electric resistance of concrete, changes in current flow, occurrence of cracks in concrete caused by the oxidation of reinforcement, rusting of bars, and changes in bond strength between reinforcing steel and concrete.

ELECTROLYTIC CORROSION OF STEEL AND VOLTAGE OF STRAY CURRENTS

The majority of electric railroads in Japan use single aerial trolley lines, the return currents being conducted by the rails to which the cathodes of electric substations are connected.

The magnitude of stray currents depends on the supply of power by the substation, intensity of the electric currents of trains, electric resistance of rails, and the resistance of the rails to leakage of electricity to the ground as well as the electrical resistance of the ground. Steel bars in reinforced concrete, at locations influenced by stray currents, are corroded at points where the currents enter and leave the concrete. When the concrete is moist its electrical resistance is about the same as common field earth but less than that of sand or gravel. The concrete thus becomes prone to electrolysis.

The manner in which the potential (capacity to transmit electrical current) of rails or metal objects buried in earth changes is shown in Fig. 1.[1] With the neutral point in the figure as the border line, on the substation side the current leaks from the metal object to cause an anodic electrolytic action, while on the opposite side current flows into the metal object causing cathodic electrolytic action. The damage from cathodic electrolysis is a lowering of bond strength due to weakening of concrete near the steel through concentration of alkali. In anodic electrolysis the steel bars are corroded by oxidation and expand thereby rupturing the concrete.

The reason for connecting cathodes to rails at substations is to localize anodic electrolytic damage to the immediate vicinity of the substations. If anodes were connected, damage would occur near the rails throughout their length as trains passed on them.

ACI Member YASUO KONDO, professor emeritus, Kyoto University, Kyoto, Japan, where this study was performed, formerly headed the Department of Civil Engineering there. He served as professor of concrete technology for more than 35 years at Kyoto and his research activities have included varied studies of plain concrete. He also introduced into Japan the *Concrete Manual* and the *Material Testing Procedure Manual*, published by the U. S. Bureau of Reclamation, and publications of TVA, PCA, and ASTM.

AKIHIKO TAKEDA, served as an instructor at Kyoto University for 2 years and this study was performed under his supervision. He is presently in charge of the concrete research of the Meishin Highway Testing Bureau of the Japan Road Corp., Kyoto.

SETSUJI HIDESHIMA, is a concrete engineer at Nippon Soda Co. where he is responsible for the field engineering. He conducted several research projects on concrete admixtures as research fellow under Professor Kondo at Kyoto University. Some of his studies were published in *Proceedings*, Japan Cement Engineering Association, 1955-1957.

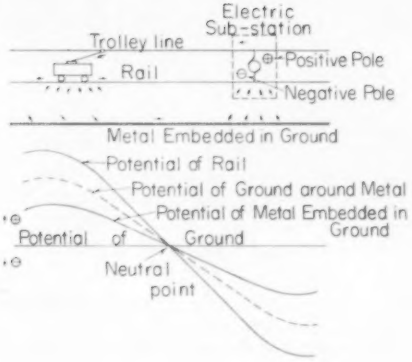

Fig. 1—Potential distribution of stray electric current

As mentioned, the possibility of electrolytic phenomena of reinforced concrete is greatly influenced by the electrical conditions of the surroundings, voltage, and voltage distribution. According to investigations made by the Railroad Research Institute of the Japanese National Railways, measurement of 24-hr potential of the ground in the Kanda substation district, Tokyo, showed that the average is, with the exception of the dead of night, more than 10 v and a maximum of 24 v.[2] The average for one year was 18 v. In other observations, approximate potentials such as 5 v for the Tokyo substation district and 30 v for the Haramachida station have been measured. There has been a case when as much as 100 v was measured at the time a train began to move.

Considering the electrical system of the potential between rail and earth, the voltage at steel bars and concrete surfaces may have a maximum of about 1.5 v and an average of 0.5 v when ties and ballast are new and the electric resistance high. However, when the structure is as old as 10 years, it can be assumed that the resistance has decreased to one-tenth or less, and the voltage between steel and concrete is about 5 v.

Given a potential between rail and earth of 50 v, distribution of this potential is considered to be as shown in Fig. 2. It can be seen that when ties and ballast are new the reinforced concrete is charged with only about 1.7 v even when potential between rail and earth is 50 v, while after many years this increases to approximately 14 v. Therefore, in carrying out experiments on electrolysis phenomena, it was considered sufficient to consider voltages up to 20 v.

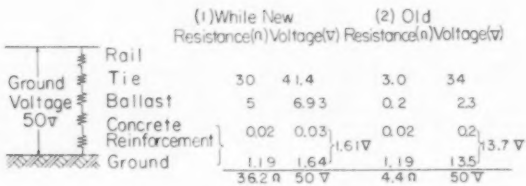

Fig. 2—Potential distribution in direction perpendicular to rail

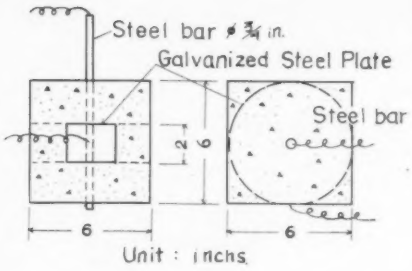

Fig. 3—Concrete specimen with protruding bar and embedded plate

TEST SPECIMENS

The problem of chemical and electrolytic corrosion of steel in concrete is a grave matter and it is thought possible that adding of calcium chloride or fly ash to concrete will accelerate electrolysis. According to the authors' experiments[3] bond is somewhat lowered by an anodic charge of 30 v when calcium chloride is used in the amount of 2 percent by weight of cement. In these experiments calcium chloride and fly ash were used individually and with a calcium lignosulfonate-type cement dispersing agent added to each. The calcium chloride was used in increments of 0, 0.25, and 1 percent while fly ash was substituted for cement in 10, 20, and 30 percent lots. The four types of concrete were charged with direct and alternating currents and observed for electrolysis.

There were three specimens made for each condition of the electrolysis tests listed in Tables 1 to 3. The term "calcium chloride concrete" in Table 1 includes concrete containing calcium chloride with and without dispersing agent; similary with the term "fly ash concrete."

Type I portland cement was used for the concrete. Fine aggregate was from the Kizu River. Coarse aggregate was from the Yasu River and graded 1-¾, ¾-⅜, ⅜-$^{3}/_{16}$ in. and combined at the ratio 3:3:4. The fly ash was from Ube. The calcium chloride was 75.5 percent pure and the calcium lignosulfonate-type dispersing agent was P–5.

The nominal cement content of the concrete was 539 lb per cu yd while the required slump was 8 in. Trial mixes were made for each type of concrete used and mix proportions, as shown in Table 4, were then decided on. Mixing time was 2.5 min and was performed with a 2 cu ft tilting-type mixer. Immediately after discharge, concrete was placed in molds and compacted with a rod-type vibrator. Fig. 3 shows the concrete specimens which were 6 x 6 x 6 in. cubes, each having an electrode consisting of an embedded steel bar and a galvanized iron sheet $^{1}/_{60}$ in. thick. Besides these, specimens with steel bars completely embedded in concrete were made, as shown in Fig. 4, to obtain results closer to practical conditions. The concrete cover was 2 in. and current was applied at the plates attached to the upper and lower surfaces.

TABLE 1—ELECTROLYSIS TESTS OF CALCIUM CHLORIDE CONCRETES BY APPLICATION OF DIRECT CURRENT

Amount of admixture, percent		Direct current, volts			
$CaCl_2$	Calcium lignosulfonate	0	5	10	20
0	0	No current	+	+	+
0.25	0	No current	+	+	+
1.00	0	No current	+	+	+, −*
0	0.25	No current	+	+	+
0.25	0.25	No current	+	+	+
1.00	0.25	No current	−	−	+, −*

*+, Anodic connection; −, Cathodic connection

TABLE 2—ELECTROLYSIS TESTS OF FLY ASH CONCRETES BY APPLICATION OF DIRECT CURRENT

Amount of admixture, percent		Direct current, volts			
Fly ash	P-5	0	5	10	20
10	0	No current	+	+	+
20	0	No current	+	+	+, −*
30	0	No current	+	+	+
10	0.5	No current	+	+	+
20	0.5	No current	+	+	+, −*
30	0.5	No current	+	+	+

*+, Anodic connection; −, Cathodic connection

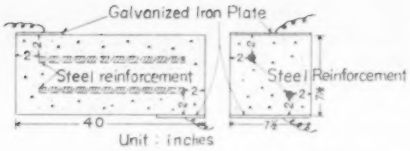

Fig. 4—Concrete specimen with completely embedded steel reinforcement

TABLE 3—ELECTROLYSIS TESTS BY APPLICATION OF ALTERNATING CURRENT

Type of concrete	0 v	20 v
Plain	No current	Current applied
0.25 percent $CaCl_2$	No current	Current applied
1 percent $CaCl_2$ + 0.25 percent calcium lignosulfonate	No current	Current applied
20 percent fly ash substituted	No current	Current applied
20 percent fly ash + 0.5 percent P-5	No current	Current applied

After specimens were cast they were covered immediately with wet straw mats. On stripping, they were placed in a 21 C water curing tank for 5 days and then immediately given three coats of a membrane curing compound to prevent evaporation of moisture. They were then kept in a room of constant temperature and humidity and moist-cured for 7 days before charging with electricity.

METHOD OF EXPERIMENTS

In applying either direct or alternating current, only the source was differed and the current was carried by the same wiring system. Fig. 5 shows the schematic arrangement while Fig. 6 gives a view of specimens being tested. In causing anodic electrolysis the steel bars were connected to the anode and the galvanized iron plates were connected to the cathode, whereas in cathodic electrolysis the connections were reversed. Both direct and alternating currents were applied to the specimens with protruding bars 3 hr a day for 14 days. Control specimens for comparison were kept under the same conditions except that no electric current was applied. The specimens with the completely embedded steel bars were subjected to 20 v dc 3 hrs a day for 28 days.

TABLE 4—MIX PROPORTIONS OF CONCRETE

Type of test	Maximum size aggregate, in.	Range of slump, in.	Unit water content, lb per cu yd	Cement factor, lb per cu yd	Unit fly ash content, lb per cu yd	Water-cement ratio, percent*	Sand-aggregate ratio, percent	Unit sand content, lb per cu yd	Unit coarse aggregate content, lb per cu yd	Unit admixture content, lb per cu yd		
										$CaCl_2$†	Calcium lignosulfonate	P-5
Calcium chloride concretes	1	7–8	315	539	—	58.2	42	1274	1807	—	—	—
	1	7–8	315	539	—	58.2	42	1274	1807	1.35	—	—
	1	7–8	315	539	—	58.2	42	1274	1807	5.39	—	—
	1	7–8	265	539	—	49.1	40	1238	1906	—	1.35	—
	1	7–8	265	539	—	49.1	40	1238	1906	1.35	1.35	—
	1	7–8	265	539	—	49.1	40	1238	1906	5.39	1.35	—
Fly ash concretes	1	7–8	297	485	54	55.0	42	1311	1861	—	—	—
	1	7–8	297	431	109	53.8	42	1309	1857	—	—	—
	1	7–8	285	378	162	52.8	42	1308	1854	—	—	—
	1	7–8	275	485	54	50.9	41	1267	1874	—	—	2.70
	1	7–8	270	431	109	50.0	41	1264	1869	—	—	2.70
	1	7–8	265	378	162	49.1	41	1261	1864	—	—	2.70
	1	7–8	315	539	—	58.2	42	1274	1807	—	—	—
	1	7–8	315	539	—	58.2	42	1274	1807	1.35	—	—
	1	7–8	265	539	—	49.1	40	1238	1906	5.39	1.35	—
	1	7–8	297	431	109	53.8	42	1309	1857	—	—	—
	1	7–8	270	431	109	50.0	41	1264	1869	—	—	2.70

*Water-(cement + fly ash) ratio in concrete containing fly ash.
†75.5 percent pure calcium chloride used.

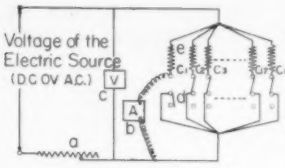

Fig. 5—Schematic diagram of wiring for electrolysis experiments

While the current was on, the voltage was kept as constant as possible and the flow of electricity was measured every hour. After completion of the 14 or 28 day periods the specimens, including the control cubes, were tested the next day for bond and slippage. Bond tests were conducted in accordance with ASTM C 234-54. Those specimens which had been tested for bond were then broken to observe the condition of the corroded steel.

OBSERVATIONS ON RESULTS

Changes in resistance of specimens during current application

Although the type and amount of admixture used in the concrete differed, curing and method of applying current were identical so that when the voltage was constant the amount of electricity consumed and the variation in the electrical resistance could be regarded as indicating the variation in moisture content, temperature of the concrete (with 30 v dc the temperature rise was approximately 3 C for anodic charges and 1 C for cathodic charges), and rusting and concentration of gas around the steel bars caused by electrolysis.

Of these factors the first two were not important as the experiments were conducted in a constant moisture and temperature room and the specimens sealed with membrane curing compound. The amount of electricity and change in resistance could be thought to be an indication of the progress of electrolysis. As it was impossible to avoid slight changes in voltage and distance between poles, the first readings in resistance of the materials used was taken as a basis for comparison. Results were as follows:

1. The ratio of increase in resistance for 20 v dc anodic connections is shown in Fig. 7 and 8. Though the data spread was only about ± 5 percent from the average values plotted, it is clearly shown that for the calcium chloride concretes at 1 percent addition the increase suddenly became intensified after 7 days, while at 0.25 percent there was not so marked a difference from the plain concrete. When calcium lignosulfonate was used with 1 percent calcium chloride the amount of change in the ratio of increase was reduced considerably. For fly ash concretes

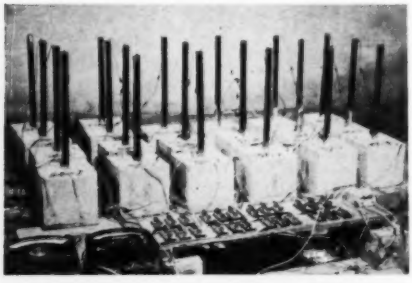

Fig. 6—Test setup for dc electrolysis

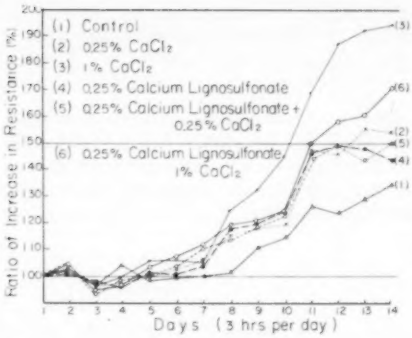
(1) Control
(2) 0.25% $CaCl_2$
(3) 1% $CaCl_2$
(4) 0.25% Calcium Lignosulfonate
(5) 0.25% Calcium Lignosulfonate + 0.25% $CaCl_2$
(6) 0.25% Calcium Lignosulfonate, 1% $CaCl_2$
Ratio of Increase in Resistance (%)
Days (3 hrs per day)

Fig. 7—Duration current applied versus ratio of increase in resistance for $CaCl_2$ concrete (20 v dc)

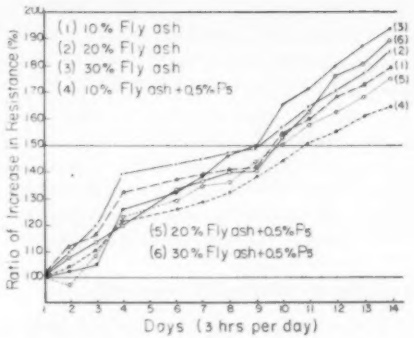
(1) 10% Fly ash
(2) 20% Fly ash
(3) 30% Fly ash
(4) 10% Fly ash + 0.5% Ps
(5) 20% Fly ash + 0.5% Ps
(6) 30% Fly ash + 0.5% Ps
Ratio of Increase in Resistance (%)
Days (3 hrs per day)

Fig. 8—Duration current applied versus ratio of increase in resistance for fly ash concrete (20 v dc)

the ratio of increase was not as conspicuous, although for 30 percent substitution both with and without calcium lignosulfonate there was a comparatively large increase towards the end. Again, the increase was less for the concrete with calcium lignosulfonate added.

2. In the case of anodic electrolysis with 10 v dc the ratios of increase in resistance were all small, being approximately one-half of the 20-v tests. The results for calcium chloride concrete were the opposite of the 20-v tests and plain concrete showed the greatest increase. It was therefore surmised that addition of 1 percent calcium chloride did not increase electrolysis at 10 v dc. The decrease with the added use of calcium lignosulfonate was the same as at 20 v. For fly ash concretes the trend was the same as at 20 v, but with smaller increases in resistance.

3. For 5 v dc anodic connections the ratio of increase was even smaller and no effect of admixtures on electrolysis could be seen.

4. In the case of 20 v dc and cathodic connection there was considerable change in resistance during the 3 hr of current passage every day. This tendency was especially notable in the latter half of the testing period. This is because hydrogen gas forms around the surface of the cathodes, or steel bars, and is stored there by reduction. The resistance dropped on turning off the current at end of the daily 3 hr to a point where the next day's passage of current began, but it recovered slightly higher than at the beginning of the previous day. Calcium chloride concrete shows greater change in resistance than fly ash concrete, probably because of higher content of electrolytic substances. An example of the relation between the ratio of increase in resistance and the number of days of current application is shown in Fig. 9.

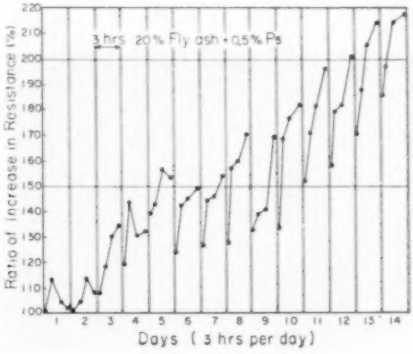

Fig. 9—Ratio of increase in resistance versus duration of current application for cathodic electrolysis

5. When 20 v ac was used there was no noticeable change in resistance as with 20 v dc and there was no specical phenomena seen at the electrodes. The increase in resistance was greatest for plain concrete and so far as this investigation is concerned, there was no effect on electrolysis by the type or amount of admixture used.

6. For concrete containing 1 percent calcium chloride with completely embedded steel bars, as shown in Fig. 4, the current applied was 20 v dc 3 hr a day for 28 days and it was thought that the two steel bars placed between the electrode plates would be affected by anodic and cathodic electrolysis resulting after a few days in a change of potential. Actually, however, the amount of electricity passing through the concrete was extremely little and there was no rusting of steel or cracking of concrete seen after 28 days.

Rusting of steel reinforcement

After completing passage of current and bond tests, the specimens were broken and inspected for rusting. Rust was first completely removed to expose the black surface of the bars, then bathed in alkali to remove all oils. For 5 v dc anodic connection, 20 v dc cathodic connection, and 20 v ac, there was no passage of electricity therefore no changes in the steel, but for 10 and 20 v dc anodic connections there was some forming of rust even for plain concrete with increased formations in proportion to the amount of calcium chloride or fly ash used. Using 1 percent calcium chloride or 30 percent fly ash substitutions, rust formed in thin films over the entire surface of the steel. However, this rusting was lessened by the additional use of calcium lignosulfonate. Generally speaking, the calcium chloride concretes showed greater rusting of steel than the fly ash concretes.

Cracking of concrete due to electrolytic corrosion

The only concrete which cracked was subjected to 20 v dc and contained 1 percent calcium chloride with steel reinforcement acting as anodes. The cracking is explained by the anodic electrolysis of the steel in which inorganic acid and oxygen form at the face of the steel and cause oxidization. As oxidization progresses the steel rusts and expands. This occurs most often during the passage of current when there is a rapid increase in resistance. A day or two after cracking, the increase becomes much less rapid. The cracking is shown in Fig. 10.

Fig. 10—Cracking of concrete in specimen with anodic connection, 1 percent $CaCl_2$ (20 v dc)

TABLE 5—INDEX OF INCREASE IN BOND STRENGTH AT 2.4 × 10^{-3} IN. SLIPPAGE OF VARIOUS CONCRETES WITH NO APPLICATION OF CURRENT

	Plain (control)	0.25 percent $CaCl_2$	1 percent $CaCl_2$
Bond, psi	423.8	445.2	503.5
Index of increase	100	105	119
Admixture	10 percent fly ash	20 percent fly ash	30 percent fly ash
Bond, psi	452.3	415.3	391.1
Index of increase	107	97	92
Admixture	0.25 percent calcium lignosulfonate	0.25 percent $CaCl_2$ + 0.25 percent calcium lignosulfonate	1 percent $CaCl_2$ + 0.25 percent calcium lignosulfonate
Bond, psi	470.8	355.6	551.9
Index of increase	111	117	130
Admixture	10 percent fly ash + 0.5 percent P-5	20 percent fly ash + 0.5 percent P-5	30 percent fly ash + 0.5 percent P-5
Bond, psi	477.9	431.0	406.8
Index of increase	113	102	96

Bond of specimens with no current passage

In Fig. 11 and 12 the bond-slippage values of concretes containing calcium chloride, fly ash, and calcium lignosulfonate individually or in combinations are shown. Bond of calcium chloride concretes and fly ash concretes at 2.4 × 10^{-3} in. slippage is given in Table 5 compared with that of control specimens. As can be seen from this table, bond increases in proportion with the amount of calcium chloride used. This tendency is further enhanced with the added use of calcium lignosulfonate. It can be seen that the ratio of increase in bond of the two admixtures combined is the sum of the two used individually. When fly ash is used the bond becomes poorer as the percentage of substitution increases.

Change in bond of calcium chloride concrete on passage of direct current

The bond-slippage relations of calcium chloride concretes subjected to 0, 5, 10, and 20 v dc with the steel bars connected to act as anodes and those subjected to 20 v dc with cathodic connections are shown in Fig. 13 to 16. Curves for 5 v dc, however, were no different from that of 0 v and were thus

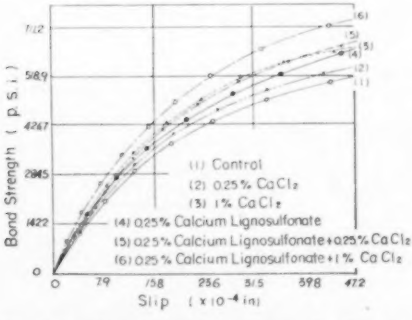

Fig. 11—Bond strength versus slip for no application of current in $CaCl_2$ concrete

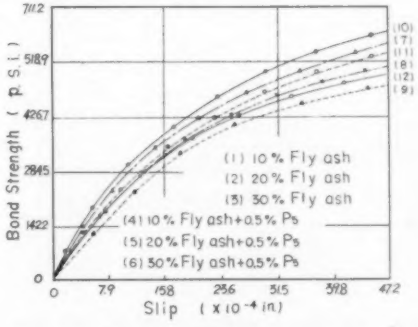

Fig. 12—Bond strength versus slip for no application of current in fly ash concrete

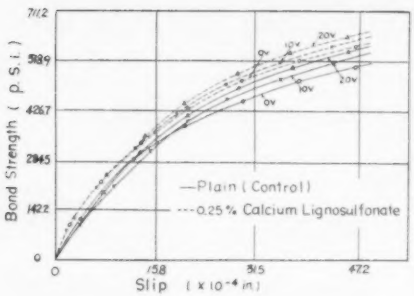

Fig. 13—Bond strength versus slip for direct current and anodic connection in control specimens

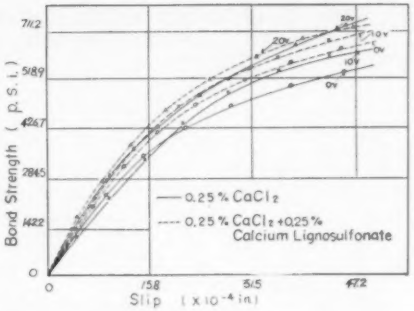

Fig. 14—Bond strength versus slip for direct current and anodic connection in concrete with 0.25 percent $CaCl_2$

omitted. The bond increases for anodic passage of current. This is thought to be due to anodic electrolysis, formation of rust increasing friction, and the raising of concrete temperature by the electricity which accelerates hardening. The increase in bond on application of current was not considered a direct indication of the degree of rusting. In Table 6 a comparison is made of the bond of various concretes at a slippage value of 2.4×10^{-3} in.

There was no change in bond of calcium chloride concretes subjected to 5 v dc anodic and 20 v cathodic current passage, while with 10 and 20 v dc anodic connections there were bond increases of 3 to 6 percent and 2 to 10 percent, respectively. In other words, in concrete containing 1 percent or less calcium chloride, direct passage of direct current 20 v or less will not lower bond through electrolytic corrosion, but rather, increases bond.

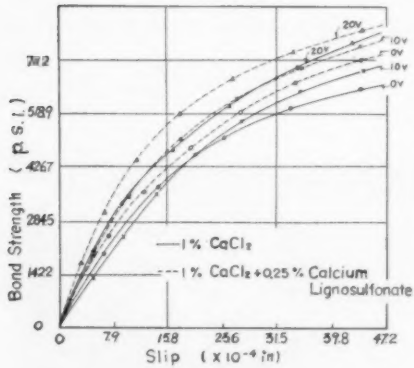

Fig. 15—Bond strength versus slip for direct current and anodic connection in concrete with 1 percent $CaCl_2$

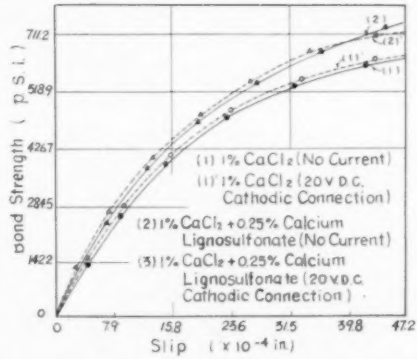

Fig. 16—Bond strength versus slip for direct current and anodic connection in $CaCl_2$ concrete

TABLE 6—INDEX OF INCREASE IN BOND STRENGTH AT 2.4 × 10^{-3} IN. SLIPPAGE OF CALCIUM CHLORIDE CONCRETE

Admixture	Plain (control)			0.25 percent $CaCl_2$			1 percent $CaCl_2$			
Voltage	0	10	20	0	10	20	0	10	20	−20
Bond, psi	423.8	435.2	462.2	445.2	475.0	519.1	503.5	519.1	590.2	507.8
Index of increase	100	100	100	105	109	128	119	119	128	

Admixture	0.25 percent calcium lignosulfonate			0.25 percent $CaCl_2$ + 0.25 percent calcium lignosulfonate			1 percent $CaCl_2$ + 0.25 percent calcium lignosulfonate			
Voltage	0	10	20	0	10	20	0	10	20	−20
Bond, psi	470.8	475.0	497.8	497.8	522.0	547.6	551.9	590.2	648.5	557.5
Index of increase	111	109	108	117	120	119	130	136	140	

TABLE 7—INDEX OF INCREASE IN BOND STRENGTH AT 2.4 × 10^{-3} IN. SLIPPAGE OF FLY ASH CONCRETE

Admixture	10 percent fly ash			20 percent fly ash				30 percent fly ash		
Voltage	0	10	20	0	10	20	−20	0	10	20
Bond, psi	456.6	476.5	524.8	418.1	433.8	492.1	416.7	391.1	409.6	460.8
Index of increase	108	110	114	99	100	107		92	94	100

Admixture	10 percent fly ash + 0.5 percent P-5			20 percent fly ash + 0.5 percent P-5				30 percent fly ash + 0.5 percent P-5		
Voltage	0	10	20	0	10	20	−20	0	10	20
Bond, psi	480.7	497.8	541.9	491.0	452.3	503.5	439.5	409.6	429.5	477.9
Index of increase	113	114	117	102	104	109		97	99	93

Change in bond of fly ash concretes subjected to passage of direct current

The bond-slippage relations of fly ash concretes subjected to 0, 5, 10, and 20 v dc with anodic connections and those subjected to 20 v dc with cathodic connections are shown in Fig. 17 to 20. In this case also there was no difference between 0 and 5 v. Table 7 shows a comparison of bond for various concretes when the slippage value is 2.4 × 10^{-3} in.

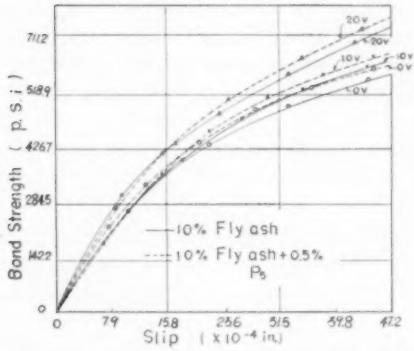

Fig. 17—Bond strength versus slip for direct current and anodic connection in concrete with 10 percent fly ash

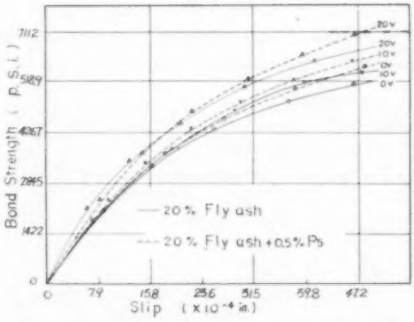

Fig. 18—Bond strength versus slip for direct current and anodic connection in concrete with 20 percent fly ash

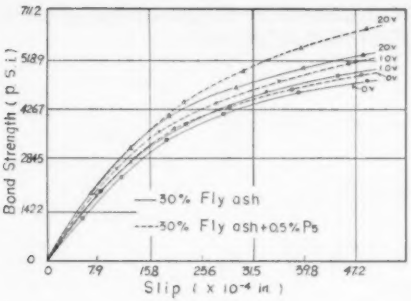

Fig. 19—Bond strength versus slip for direct current and anodic connection for concrete with 30 percent fly ash

As can be seen from the table, fly ash concretes show no change in bond to steel when subjected to 5 v dc anodic and 20 v dc cathodic connections while for the 10 and 20 v anodic connections the bond increases by 4 to 5 percent and 13 to 18 percent, respectively.

Change in bond of specimens subjected to alternating current

The bond-slippage relations and comparison of bonds at the slippage value of 2.4×10^{-3} in. of various concretes subjected to 20 v ac are shown in Fig. 21 and Table 8, respectively. From these it can be seen that the bond strengths of the various concretes are higher than for direct current, but this is because fresh cement was used in the concrete for these tests.

There was no change due to electrolysis in bond of concretes subjected to 20 v ac as there was in 20 v dc and the variation was exactly the same as for concretes not subjected to electric current. Therefore, it was concluded that there is no effect on bond to steel, even for 20 v charges, when alternating current is used.

It was postulated that when alternating current was applied, because of the difference in material of steel, cement paste, and aggregates, there would occur rectification phenomena in the current flowing down the interface of steel and concrete so that over a long period of passage there would occur electrolysis similar to that caused by direct current. However, when the electric waves between the electrodes and the concrete surface were observed by a Brown

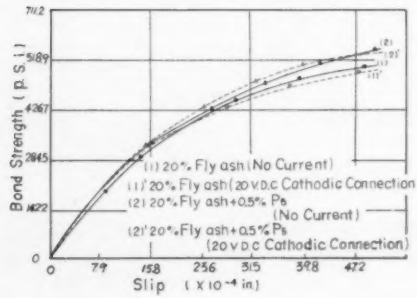

Fig. 20—Bond strength versus slip for concrete with 20 percent fly ash, with and without current, and with and without

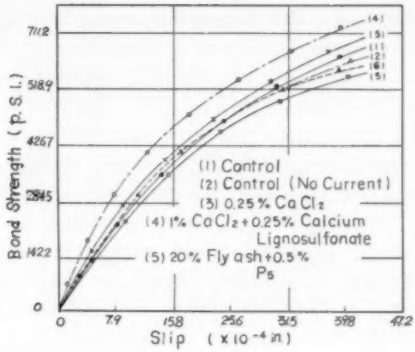

Fig. 21—Bond strength versus slip for 20 v ac comparing control specimens and concretes with admixtures

tube oscillograph no difference was found between them and the waves of the power source.

CONCLUSION

The experiments described in this paper were conducted to investigate the electrolytic corrosion phenomena occurring when outside electric current is applied to reinforced concrete using calcium chloride, fly ash, and calcium lignosulfonate, singly or in combination, as admixtures. The following conclusions are drawn from these experiments.

1. There is no effect from currents up to 5 v dc when using calcium chloride or fly ash as admixtures where parts of the reinforcing bars are exposed and outside direct current is applied directly to the bars in such a way that they act as anodes. At 10 v dc, when 1 percent and more calcium chloride or 30 percent and more fly ash are used, the entire surface of the parts of reinforcing bars embedded in concrete show light rust, but no cracking occurs and bond of steel to concrete is increased by 4 to 10 percent. With the same amount of calcium chloride or fly ash at 20 v dc, the rusting of bars embedded in concrete differs depending on the amounts of admixture used, in cases being comparatively pronounced, and cracking of concrete occurs when the amount of calcium chloride used exceeds 1 percent. Overall bond of steel to concrete is not lowered and rather it is increased by 13 to 18 percent. When the reinforcing bars act as cathodes or when alternating current is applied no electrolysis occurs at voltages under 20 v.

2. In concrete containing 1 percent calcium chloride where reinforcing bars are completely embedded in con-

TABLE 8—INDEX OF INCREASE IN BOND STRENGTH AT 2.4 × 10^{-3} IN. SLIPPAGE OF VARIOUS CONCRETES SUBJECTED TO 20 V ALTERNATING CURRENT

Admixture	Bond, psi	Index of increase
Plain (no current)	492.1	100
Plain	648.6	102
0.25 percent $CaCl_2$	527.7	107
1 percent $CaCl_2$ + 0.25 percent calcium lignosulfonate	584.6	119
20 percent fly ash	477.9	97
20 percent fly ash + 0.5 percent P-5	503.6	102

crete and the distance from surface to bar is 2 in. or more, no rusting of steel or cracking is seen even when 20 v dc is applied to the concrete surface.

3. When the properties of concrete containing 1 percent calcium chloride or 30 percent fly ash are improved by the addition of 0.25 percent calcium lignosulfonate, the corrosion phenomena of bars acting as anodes, occurring when direct current is applied, are greatly reduced.

It appears that the damage from electrolytic corrosion in reinforced concrete containing calcium chloride or fly ash as admixtures is generally overestimated. The greatest cause of such damage is current straying from railroad tracks, the voltage reaching concrete nearby being about 14 v when ties and ballast are approximately 10 years old and electrical resistance is weakened. However, when steel bars are not exposed from the concrete no corrosion can be seen although 1 percent calcium chloride is used and as much as 20 v dc is applied.

Usually, the actual voltages of currents reaching concrete in the field are considerably lower than the 20 v used in these experiments and furthermore they occur off and on over a long period. It may be assumed that reinforced concrete containing calcium chloride or fly ash in moderate quantities can be amply protected by improving properties of the concrete by addition of calcium lignosulfonate and through observation of proper concreting practices.

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Title No. 56-22

Comparison of Measured and Calculated Stiffnesses for Beams Reinforced in Tension Only

By BILL G. EPPES

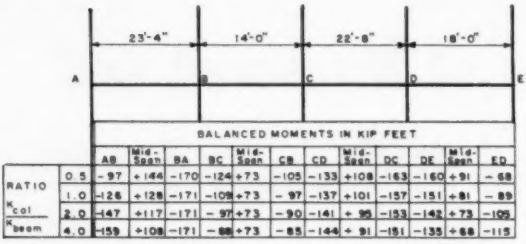
In any study of mechanics of indeterminate structures stiffness plays an important role in analysis. Analysis leads to the determination of the redundant quantities, which in turn lead to the determination of the critical bending moments and shears. The factor EI, which is a measure of stiffness, is necessary for determining deflections of any elastic structure, be it statically determinate or statically indeterminate.

This paper is a presentation of the findings of an experimental measurement of stiffness (EI) for reinforced concrete beams. Measured values of stiffness are compared with the calculated values for nine beams.

It was found in this study that the measured values of stiffness decrease with increased bending moment, showing first a large value for stiffness for the lower values of moment and then a rather sudden transition to a lower value of stiffness for larger values of moment. The larger measured values compare reasonably well with the calculated gross section of a reinforced concrete beam with transformed area of steel and the lower values compare fairly well with the calculated net section of the reinforced concrete beam with transformed area of steel included.

■ SOLUTIONS TO THE REDUNDANT QUANTITIES OF A STATICALLY INDETERMINATE FRAME require equations in addition to those of statics. These equations are based on the behavior of the frame under load. Contained in these equations are properties of the frame or its members, specifically EI (shown here in combination as it usually appears). Of particular concern is I, since this is the quantity that usually varies while E remains constant. The use of E, of course, indicates that a frame analysis is based on ideas of elasticity. The term EI always appears in equations for determining the elastic deflection of members.

The designer is frequently confronted with the need for a value of I for use in frame analysis. When analyzing moments in a frame the ratios of stiffness play the predominant role in the final determination of moments. A useful demonstration of the difference in the final balance of moments as



BALANCED MOMENTS IN KIP FEET

$\frac{K_{col}}{K_{beam}}$ RATIO	AB	Mid-Span	BA	BC	Mid-Span	CB	CD	Mid-Span	DC	DE	Mid-Span	ED
0.5	-97	+144	-170	-124	+73	-105	-133	+108	-163	-160	+91	-68
1.0	-126	+128	-171	-109	+73	-97	-137	+101	-157	-151	+81	-89
2.0	-147	+117	-171	-97	+73	-90	-141	+95	-153	-142	+73	-109
4.0	-159	+108	-171	-88	+73	-85	-144	+91	-151	-135	+68	-115

Fig. 1—Variation in stiffness affecting moments in beams

changes occur in the stiffness ratios between beams and columns is given in the Portland Cement Association's *Continuity in Concrete Building Frames*,[1]

> The effect of variations in stiffness is illustrated . . . for the following ratios of column to beam stiffness: ½, 1, 2, and 4. The tabulated values indicate that some moments, especially those in exterior spans, are sensitive to changes in column stiffness, whereas others are not. It is advisable not to ignore this question but to be sure that appropriate stiffness values are used in the analysis.

Shown in Fig. 1 is a copy of Fig. 17 of Reference 1. (K is used to represent the flexural stiffness of the member.*)

If the value of stiffness used when designing a beam in a frame were higher than the true stiffness, the ratio K_{col}/K_{beam} would be too low and some of the calculated moments would be different from the actual moments. If the ratio between the design moment of inertia and the actual moments of inertia were 2:1 then the resulting ratio of K_{col}/K_{beam} would be increased from 0.5 to 1.0 and the bending moment in the exterior span on the left side would increase from −97 kip-ft to −126 kip-ft. This means that the actual moment is 30 percent higher than the designer's value of moment, as determined from conventional methods.

Importance of study

The methods given in most published material on reinforced concrete concerning a method of determining the moment of inertia of concrete sections are rather varied. Although it is well known that reinforced concrete is not a solid homogeneous material, it is quite often treated as such by designers when computing moment of inertia.

> Opinions differ as to the proper method of computing moment of inertia . . . The preponderance of opinion is in favor of computing the moment of inertia for the concrete section only by considering it as a homogeneous section and neglecting the reinforcement. This method is recommended by the authors.[2]

The *CRSI Design Handbook*[3] recommends that:

> The moment of inertia of the cross section can be obtained in one of two ways:
> (a) When making preliminary computations, use the moment of inertia of the gross outline of the concrete section, omitting the reinforcing steel. If the same method is

*$K = EI/L$, where E is the modulus of elasticity of the member, I is the moment of inertia of the cross section of the member, and L is the length of the member measured from the center lines of the members which it intersects.

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applied to all members of a frame, this is fairly reasonable. In an ordinary continuous tee-beam, there is a tee section near midspan and a rectangular section at either end, and the point of inflection moves considerably under different loading conditions, so it is clear why a high degree of precision is not obtainable.

(b) Some designers, feeling that a heavily reinforced beam should have more resistance than a lightly reinforced one, take the moment of inertia of the transformed area, though there is a question as to whether this is any more precise, and it can only be done after considerable preliminary designing to establish all the needed data.

Here the author seems to be uncertain as to how the moment of inertia of concrete sections should be treated; there is also a question concerning the precision of moment of inertia calculations.

The Joint Committee Report[4] recommends that:

> The moment of inertia of flexural members, for purposes of computing the relative stiffness, may be that of the gross cross-sectional area of the member, making reasonable allowance for the effect permitted in 804(d). (Refers to allowable flange width.) For columns or other compression members the transformed area of the steel reinforcement should be included.

The *British Standard Code of Practice for Reinforced Concrete* states in Article 506:

> For the purpose of estimating the stiffnesses of a reinforced concrete member the moment of inertia may be calculated on its whole section. Allowance for the reinforcement may be made using the appropriate modular ratio. In the case of a beam the breadth of the compression slab shall be taken in accordance with clause 601(g). (Refers to the allowable flange.) The method employed in estimating moment of inertia shall be the same for all members considered in any one calculation.

The above article is explained in *Explanatory Handbook on the Code of Practice for Reinforced Concrete:*[5]

> It will be noted from clause 506 that the moment of inertia and therefore the stiffness of members may be calculated by one of three methods, provided the same method is used throughout the calculation. The methods are:
>
> (a) Using the whole area of the section, ignoring the steel reinforcement
>
> (b) As (a) but including the steel at its appropriate modular ratio
>
> (c) Using the compression area of the concrete, ignoring the tension area, and allowing for the steel at its appropriate modular ratio
>
> These will generally lead to different results, although in normal cases the difference is not likely to affect design considerably.*

*Reference 5, p. 98.

SOURCE	RECOMMENDATIONS FOR CALCULATED MOMENT OF INERTIA					
	gross section		gross section with transformed area of steel included		cracked section with transformed area of steel included	
	beam	col.	beam	col.	beam	col.
Taylor, Thompson and Smulski	X	X				
CRSI Design Handbook	X	X or	X	X		
Joint Committee Report	X			X		
British Standard Code	X	X or	X	X or	X	X
ACI Code	X					
	Any reasonable assumption may be adopted for computing relative stiffness of columns and of floor systems. The assumption made shall be consistent throughout the analysis.					

Fig. 2—Various recommended methods of determining moment of inertia

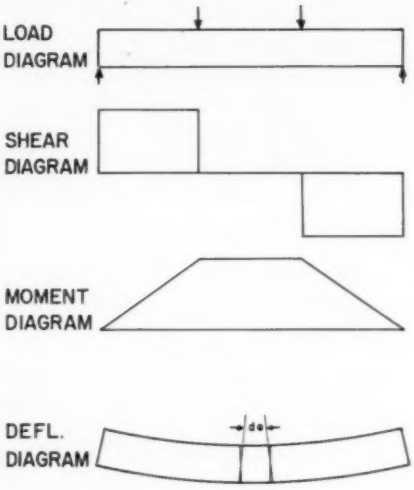


Fig. 3—Symmetrical loading used to produce a constant moment for a finite length of beam

The British, then, allow the designer to take his choice as to the method he desires for the computation of moment of inertia. Further lack of agreement concerning moment of inertia and stiffness is expressed in Reference 1.

> Whatever type of analytical method the designer may use, it is necessary to establish the relative stiffness of all beams and columns. Stiffnesses are functions of cross-sectional dimensions, but these are not known at the outset and must therefore be estimated. Another source of trouble is that the calculation of moment of inertia is often slow and is based on certain assumptions about which complete agreement has not yet been reached . . .
>
> For beams, the question arises regarding the effect of flange upon the stiffness. Both the ACI Code and the Joint Committee Report recommend that I be based upon gross section but whereas no allowance is made for reinforcement, allowance should be made for the effect of the flange.*

A study of the above quotations will show that there is a general lack of agreement concerning an absolute method for determining moment of inertia and, further, that one may well take his choice as to how he calculates the stiffness of various members of a frame, depending on whose recommendations he follows (Fig. 2). It then becomes important for the designer to know the limitations of the various methods for the handling of moment of inertia and how the calculated values compare to the actual values. The purpose of this paper is to present measured values of I and compare them with the usual calculated values.

*Reference 1, pp. 15, 18.

Theoretical considerations

The experimental measurement of EI is based on the equation $d\Theta = Mdl/EI$. Of course, if the moment is held constant over a finite length l, then the equation becomes $\Theta = Ml/EI$, or solving for EI the equation may be written $EI = Ml/\Theta$. To produce a constant moment over a finite length, a symmetrical loading as shown in Fig. 3 is used. The moment is determined from measurements of load and/or reaction. The length l is established using the gage length of a strain measuring device, and the angle Θ is determined by measuring the strain along the length of the beam at various depths. Various experimenters have used a technique of this type to determine Θ.

If the moduli of elasticity of the concrete and reinforcing steel are known then the value of moment of inertia can be isolated from EI. For the equation $\Theta = Ml/EI$ to be valid concerning the concrete:

(a) The stresses caused by bending are below the proportional limit, so that Hooke's law holds. Concrete does not behave strictly in accordance with Hooke's law, but it is commonly assumed in design that it does. Measurements made when either the concrete or the steel exceed the proportional limit will not be valid.

(b) A plane section across the beam remains plane after the beam is bent.

PROCEDURE

Preparation of specimens

The design of the test beams was based on the following factors:

1. It was intended that the beams not fail in bond or diagonal tension until the stresses in flexure were well above working stresses.

2. Specimens as large as possible were desired so that standard size materials could be used (1-in. maximum size aggregate and deformed bars). The specimens were limited in size to 6 x 6 x 36 in. so that standard steel beam forms could be used. A beam of this size is easily handled for instrumentation and placing in the testing machine.

3. The percentage of steel was to vary but remain in the normal range for beam reinforcing.

Two sets of beams were cast for this experiment. The designation for the first set of beams is 1 followed by the number of the beam i.e., Beam 1-1. The designation for the second set of beams is 2 followed by the number of the beam i.e., Beam 2-1. Two batches were required for each cast, due to the quantity of material involved. Two 6 x 12 in. test cylinders were taken with each casting. The reinforcing steel was supported by holes in the end forms, the reinforcing steel being longer than the beam.

Curing

After casting, the concrete was left standing for 24 hr, after which the forms were removed and the specimens placed in a moist room. The temperature of the moist room was a constant 70 F. The concrete specimens of the first casting remained for 9 days. The beams for the second casting were allowed to remain for 13 days.

Gage points

Special gage points were prepared for this test to be used with an 8 in. Berry strain gage modified.* Discs ¼ in. in diameter were obtained from punchings of 1/16-in. sheet steel.

*The Berry strain gage was modified by rounding the conical ends of the legs of the strain gage into a spherical shape.

A hole was drilled in the center of each disc with a No. 55 drill. Each disc was then carefully sandblasted, leaving the surface chemically clean, so that it would bond well with the cementing compound used.

Installation of gage points

The location of gage points on the beam is shown in Fig. 4. Gage points were installed symmetrically on both sides of the beam. Prior to installation of the discs the concrete beams were carefully cleaned in the areas where the gage points were to be placed to assure a good bonding surface. A grid was then marked off on the side of the beam and at the intersections of the grid lines, and the gage points were cemented to the surface.

Preparing the beams for test

Steel plates were grouted in plaster of Paris at the points of load and support. These plates were of such size that no failure in bearing could occur. The plates were given their final adjustment with a spirit level. The beams were tested in a 120,000 lb testing machine. For the first set of beams, Beams 1-1 to 1-4, the loading points were 10 in. apart. For the second set, Beams 2-1 to 2-5, they were 20 in. apart. It was felt that it would be desirable to have the constant moment section farther away from the load points. Two No. 2 stirrups were used at each end of the second set of beams to aid in preventing an early diagonal tension failure.

An 8 in. gage length Berry strain gage with a multiplication ratio of 5 to 1 was used to take all strain readings. The dial gage used with the strain gage was graduated in 0.001 in. divisions. Readings could usually be repeated with accuracy to the nearest $\frac{1}{3}$ of a division of the strain gage; thus, the strain could be measured with accuracy to about 0.000008 in. per in. This is an upper limit of the accuracy and was probably not obtained at all times. A deflection gage was mounted on the loading table to measure vertical deflection of the beam.

Testing

A load of 1000 lb was initially placed on the specimen and then released. (See Fig. 5 for load diagram.) A complete set of strain readings was then taken for zero load and checked

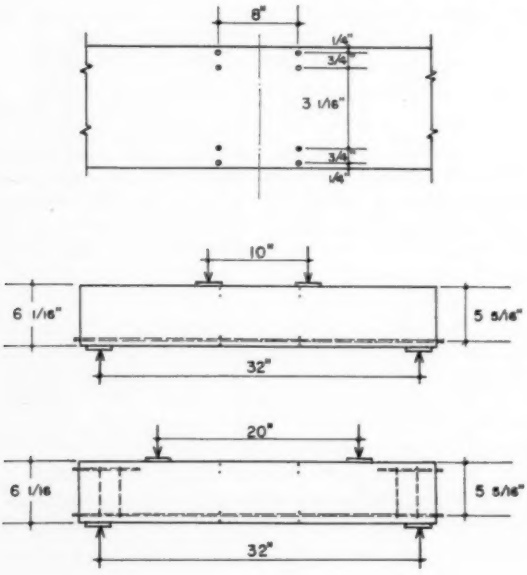
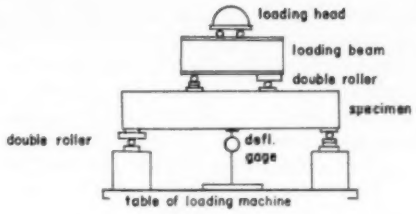


Fig. 4—Position of gage points (top), steel placing and loading for first set of beams (middle), and steel placing and loading for second set of beams (bottom)

Fig. 5—Typical loading system for beams



by an alternate reader. It was found the different readers could agree with one another to ⅓ of a division of a 0.001 in. gage. Readings were taken at predetermined regular intervals of loading so that plotted points would be evenly spaced along the final curves. The procedure then was to load the beam and take a complete set of strain readings and make observations of cracks. Cracks were detected with the unaided eye, but with the use of a strong lamp, so that the surface was well illuminated. All visible cracking was marked with a pencil at the end of each loading increment. The extreme limit of each crack was noted (See Fig. 6 and 7). The process of loading and taking strain readings and tracing cracks was continued until the beam began to experience difficulty with diagonal tension. The beams were not loaded to failure. Cylinder tests were taken immediately after the loading of the beam so that the age factor would not influence the modulus of elasticity of the concrete. The compression cylinders were tested in a 200,000 lb hydraulic testing machine. A standard 10-in. gage length was used when measuring strain. The average secant moduli of elasticity taken

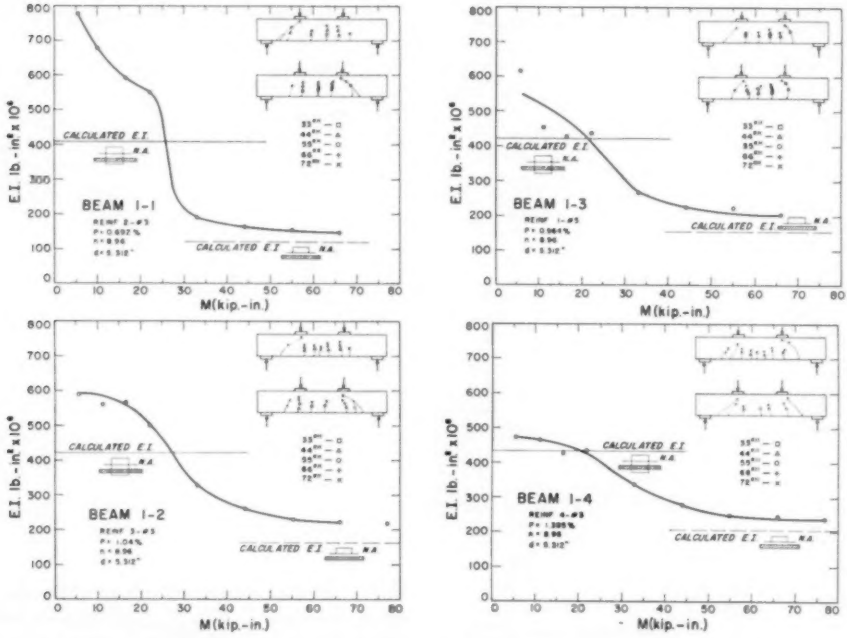


Fig. 6—Plot of measured values of *EI* and M for Beams 1-1 to 1-4. Also shown are calculated values of *EI*, the upper value was computed by the transformed section method, the lower value by the cracked section method

at 0.45 f_c' for the concrete was 3.35 x 10^6 psi for the first casting and 4.02 x 10^6 psi for the second casting. Fig. 8 shows the stress-strain curves for the two concretes and the steel.

The loading of the beam was preplanned so that several sets of readings were taken before cracking began in the beam. This was necessary so that EI could be determined for an uncracked section. As the load was increased and the beams began to crack, the depth of cracks was fairly constant at the various load increments and the spacing of cracks was somewhat regular throughout the midsection of the beam. This uniformity of cracking was probably caused by the high bond steel which tends to induce a uniform pattern of relief of the tensile strain in the concrete. As loading continued the cracking pattern was continually marked. Additional cracks opened up on occasion, but in general the initial cracks penetrated more deeply with increase in load.

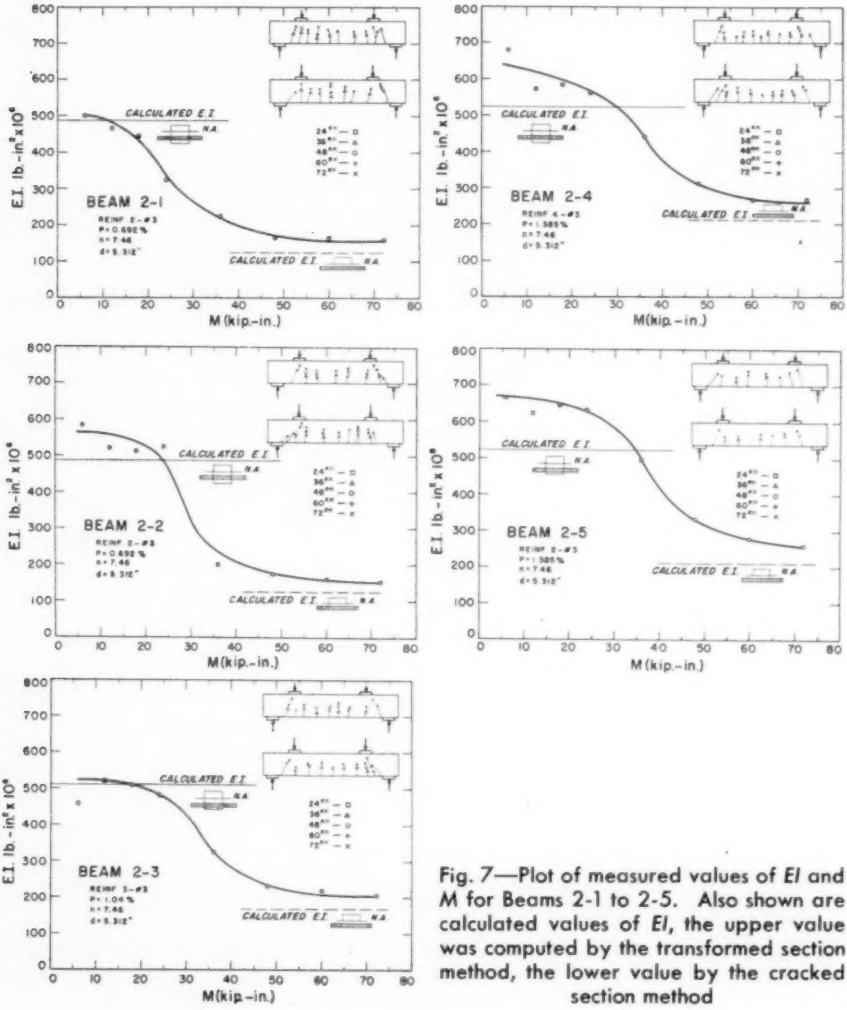


Fig. 7—Plot of measured values of EI and M for Beams 2-1 to 2-5. Also shown are calculated values of EI, the upper value was computed by the transformed section method, the lower value by the cracked section method

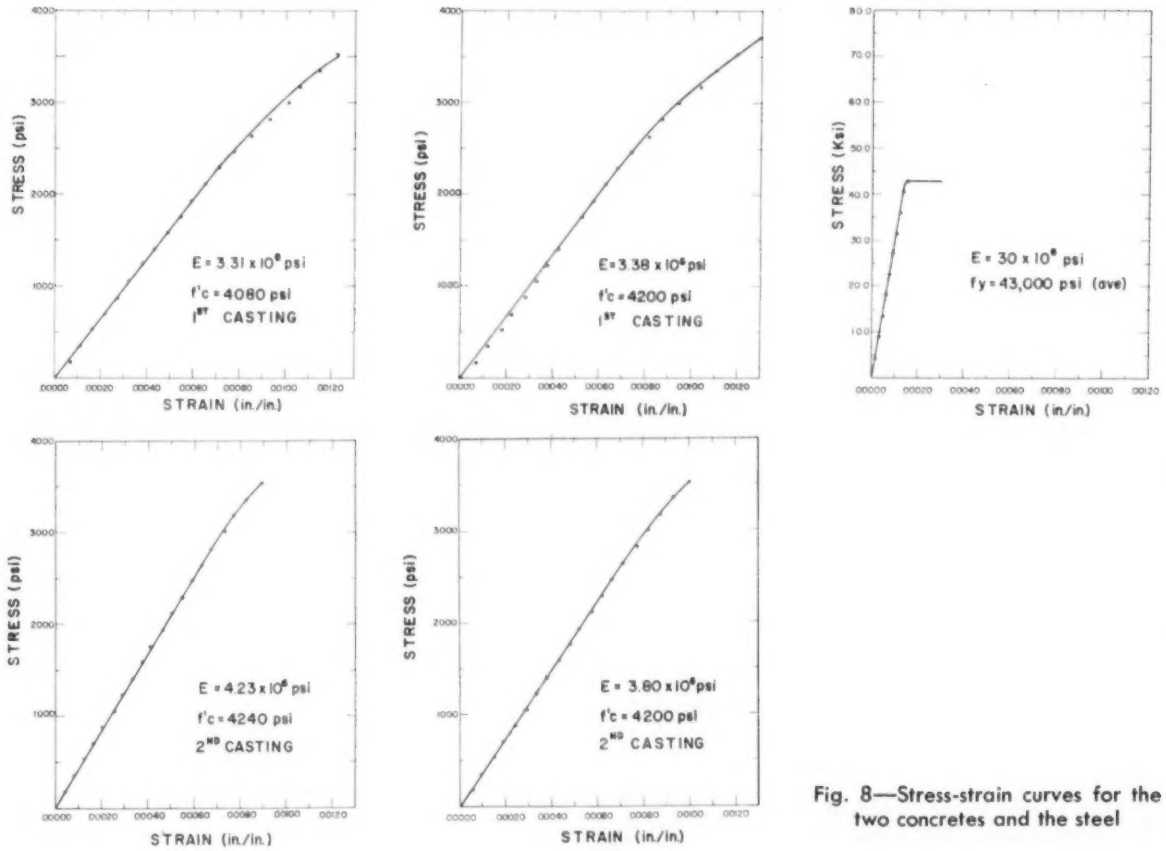


Fig. 8—Stress-strain curves for the two concretes and the steel

In plotting the strain readings as a function of depth to obtain the angle change at midspan it was noticed that the position of the neutral axis gradually crept upward as the moment increased. This in itself, that is, movement of the neutral axis, indicates a change in the moment of inertia of the midsection of the beam.

RESULTS

The stiffness of a beam is reduced as cracking is produced. In most cases when a moment crack in a beam occurs, the occurrence is sudden and the crack penetrates relatively deeply into the beam. This causes a sharp decrease in the absolute stiffness. Results from 33 reinforced concrete beams tested at the University of Illinois support this idea. In a paper by Gaston, Siess, and Newmark,[6] a portion of the presentation of their work involves a discussion of percent of maximum moment versus angle change at midspan of beam, which was plotted for each beam (Fig. 9). These beams were loaded at third points so that a constant moment section over the center portion of each beam was produced. Although most of the emphasis in these plots seems to be on a comparison of the portion of the curves at failure, an initial portion of this plot up to about $M = 0.20\ M_{max}$ shows that the initial tendency of the curves is to start at a steep slope, then at about $M = 0.20\ M_{max}$ to proceed at a flatter slope to about $M = 0.80\ M_{max}$ after which the plot flattens out rapidly. Although the information in these curves is not complete and accurate enough to closely support this thesis, the general tendency in a number of cases is as described above. This general shape of curve is

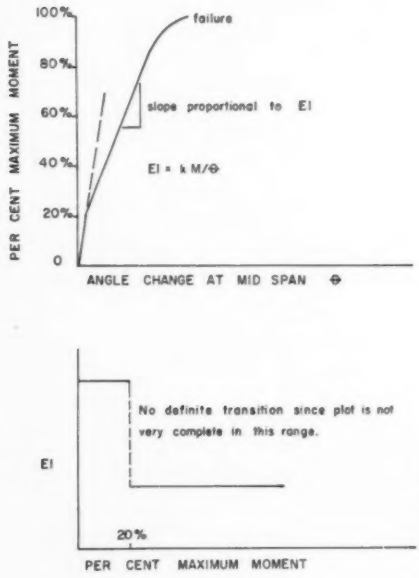


Fig. 9—Maximum moment versus angle change at midspan of beam (top); moment versus stiffness (*EI*) diagram showing reduction of stiffness for values of M greater than 0.20 M_{max} (bottom)

shown in Fig. 9. The value of EI for the curve above $M = 0.20\ M_{max}$ is estimated to be in the order of 60 percent of the initial value.

When transforming the diagram of percent M versus angle change at midspan of beam into an M versus EI diagram, the transformation becomes two horizontal lines without any definite transition between them, since EI is proportional to the value of the slope of the line in the plot of percent M versus angle change at midspan of beam ($EI = k\,M/\Theta$). (See Fig. 9.)

Fig. 6 and 7 show a plot of EI in lb-in. squared times 10^6 versus moment in kip-in. for the nine beams tested. The smooth curve connects the measured values of EI. The upper values of EI are compared with the calculated values of EI for gross section with transformed area of steel. The lower values of EI are compared with calculated values of EI for a cracked section with transformed area of steel. As one observes the curves presented, which also are plotted for varying percentage of steel, a pattern seems to be prevalent among the plots, in that the smooth curve is of Gordon-Rankine form. Further, one will note that the differences of the "high" values and "low" values of EI are farther apart, generally, for lower percentages of steel and closer together for the higher percentages of steel. This would say, then, that the stiffness of a beam for a low percentage of steel would vary more with respect to moment than for a beam with a high percentage of steel.

TABLE 1—CALCULATED DESIGN AND ULTIMATE MOMENTS

Beam No.	Mu	$Mw = \frac{Mu}{1.8}$
1–1	48.5 K-in.	26.9 K-in.
1–2	71.1 K-in.	39.6 K-in.
1–3	66.1 K-in.	36.7 K-in.
1–4	92.6 K-in.	51.3 K-in.
2–1	48.4 K-in.	26.9 K-in.
2–2	48.4 K-in.	26.9 K-in.
2–3	71.0 K-in.	39.4 K-in.
2–4	92.5 K-in.	51.2 K-in.
2–5	92.5 K-in.	51.2 K-in.

TABLE 2—VALUES OF EI FOR GROSS SECTION AND NET SECTION WITH TRANSFORMED AREA OF STEEL WHERE EI IS BASED ON $E = 1000\ f_c'$

Beam No.	EI N.A.	EI N.A.
1–1	148 x 10^6	508 x 10^6
1–2	201 x 10^6	526 x 10^6
1–3	190 x 10^6	520 x 10^6
1–4	246 x 10^6	538 x 10^6
2–1	150 x 10^6	514 x 10^6
2–2	150 x 10^6	514 x 10^6
2–3	205 x 10^6	534 x 10^6
2–4	251 x 10^6	546 x 10^6
2–5	251 x 10^6	546 x 10^6

The comparisons of calculated values of EI with the lower values of measured EI seem to form a pattern such that the differences fall in approximately the same range. However, the comparisons of the calculated values of EI for gross section with transformed area of steel compared with the higher measured values of EI seem to be more erratic. This, in part, may be attributed to experimental error. Since the experimental error was constant, the percentage error at the low values of moment would be much higher than for the higher values.

Table 1 shows values of the design moment M_w and the ultimate moment M_u, based on the equations $M_w = M_u/1.8$ and $M_u = bd^2 f_c'q\ (1 - 0.59q)$ in ACI 318-56, appendix. The values of M_w and M_u are shown on Fig. 6 and 7.

When a design moment is based on the peak volume of a moment diagram, then lower values of moment will have higher values of moment of inertia according to the distribution given by the EI versus M curve for values of moment between 0 and M_w. As the curves indicate, then, the value of moment of inertia can vary as much as 100 percent along the length of the beam as bending moment varies from zero up to the design value.

Table 2 shows values of EI for both the gross section with transformed area of steel and net section with transformed area of steel where the modulus of elasticity is based on $1000\ f_c'$. In most cases the EI value by this method shows better agreement with the experimental results than the more precise measured value of E taken from the stress-strain curve.

CONCLUSIONS

The writer does not feel that it is possible to draw formal conclusions at this time; that is, conclusions the designer may use for design information. However, certain summary statements and thoughts may be presented.

1. There appear to be two general values of EI, a value in the upper range and a value in the lower range which compare somewhat with calculated values as presented. It is well to note that the stress and strain in the concrete for the uppermost plotted point did not leave the straight line portion of the stress-strain curve.

2. It appears from the comparisons made that calculated values of EI are somewhat on the conservative side.

3. The percentage of steel reinforcement effects the range of values of EI that a beam might have.

4. Since the bending moment usually varies with respect to span, and since the EI value varies with respect to bending moment, in the process of frame analysis, one possible way of describing stiffness would be by the effective value of a member. This would require an extensive theoretical and experimental study to determine effective stiffness values.

5. Values of EI where E is based on $1000\ f_c'$ show better agreement with the experimental results than when E is taken from the stress-strain curve of

the concrete used. This appears to have some merit but may not be a good general statement since $E = 1000 f_c'$ is an approximation.

It is felt that two areas where additional studies are needed are:

1. The investigation of experimental values of EI for beams with compressive reinforcement, which the author is presently conducting.

2. Investigations of values of EI for beam columns which are cracked.

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Title No. 56-23

Rheological Behavior of Hardened Cement Paste Under Low Stresses

By J. GLUCKLICH

A set of simple static experiments with hardened cement paste is described to yield the deformational response of the material to low stresses. Using a mechanical analogy, this behavior can be described by the following elements connected in series: (a) a linear spring; (b) elements transmitting force by friction; (c) a body composed of a linear spring in parallel with a dash pot (a so-called Kelvin body); and (d) as in (c), except that the piston in the dash pot is made to move in one direction only by means of a nonreturn valve.

Alternatively, instead of (c) and (d), a body composed of a dash pot in parallel with a spring which includes slippage elements as in (b).

Creep behavior of the material is the superposition of two deformations: nonreversible creep with a relatively short time of retardation, and a reversible creep with a time of retardation one order of magnitude longer.

Quantitative results and the dependence of the various parameters on factors such as water-cement ratio and age are briefly mentioned.

■ THE OBJECT OF THE PRESENT RESEARCH was the study of the behavior of hardened cement paste under low stresses, i.e., before detectable cracking begins, and thus before the deviation from the visco-elastic relationship commences. Data were obtained with a few simple static tests along the lines of tests in 1932 by Bingham and Reiner.[1] Deviation from the test procedure of Bingham and Reiner, however, showed that one of their conclusions were erroneous. Their creep tests lasted only 7 months, and this made them attribute secondary creep to viscous flow, whereas the same test, when extended to 2 years, and correlated with a repeated loading test, reveals that the material is a solid.

The problem has generated many, and often contradictory, theories. An excellent survey of existing theories of the creep of concrete is given by Neville.[2] Since behavior of concrete is governed by many factors, the experiments reported here were performed with as many factors as possible under control. For instance, to eliminate undesirable moisture transference with the atmosphere, all specimens were coated with a thin, but effective, plastic skin. This also insured identical curing conditions for all specimens.

J. GLUCKLICH, a research fellow in the division of mechanics, Israel Institute of Technology, Haifa, is engaged in the investigation of the mechanical properties of building materials. In the last 4 years, he has been concerned mostly with concrete. His approach to basic research problems has included the use of the relatively new tool of rheology.

The results are typical of the many results obtained in the investigation, and therefore this paper bears the character of a qualitative work. This, however, is for the sake of brevity. For the same reason the influence of such factors as water-cement ratio and age on behavior are only briefly mentioned.

Bending tests were used mostly in this work since, due to rotations which superimpose distortions, the over-all deformations in such a test are greatly magnified and permit observations by simple means under low loads of short duration.

TESTS

Three series of tests (A, B, and C) were made with slender beams 83.5 x 2.27 x 2.27 cm (32.9 x 0.86 x 0.86 in.). One series (D) was made with cylinders 10.0 cm in diameter x 15.4 cm long (3.94 in. in diameter x 6.06 in. long). The specimens were of three water-cement ratios (by weight): 0.24, 0.28 and 0.32, and tests were performed at various ages to study the effect of W/C and age on the rheological properties. The results presented here will, however, be confined to the group with a W/C of 0.32 for the sake of brevity.

The cement was ordinary portland cement. Specimens were prepared, stored, and tested under controlled conditions of 16 C (61 F) and 95 percent relative humidity. They were coated immediately on demolding with a polysterene benzol solution which formed an effective skin, sealing the material from the atmosphere; thus, curing was of the retention type. Periodic weighings of the specimens showed that variations in weight in 80 weeks did not exceed 1.5 percent of the total weight. Specimens were not tested before the age of 1 month.

Test A—Instantaneous deformations

A simply supported beam was centrally point-loaded by a hanger and weights. Deflections at the center were measured by a 0.0001 in. dial gage. To exclude all time-dependent deformations, the weights were not left acting on the beams as the loading progressed, but were removed after each loading step. In fact, the procedure was to apply a load, to note immediately the deflection, and then to remove the load without delay. An increased load was then similarly applied. This procedure followed Bingham and Reiner.[1] The loading progressed up to fracture.

Test B—Time-dependent deformations

Beams were simply supported on two parallel knife-edge supports. They were centrally point-loaded by weights specially made to yield maximum bending moments of 2, 4, and 8M, where M is the bending moment produced at the center by the dead weight of the beam. There were two or three beams for each type of load and for each W/C. This loading arrangement was maintained for 2 years, and is still in progress undisturbed except for periodic deflection observations for which the specimens are transferred to the measuring device which consists of two supports of the same span with a 0.0001 in. dial gage rigidly secured to the base at half span. Means are provided to ensure exactly the same positioning of the beam on the supports for successive observations.

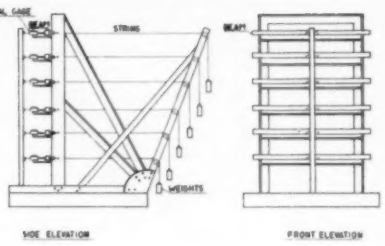

Fig. 1—Arrangement for horizontal loading of beams for Series C

Test C—Repeated loadings

In Series B, recovery on removal of load could not be observed for there was always the dead weight of the beam. In Series C, therefore, the arrangement was changed so that the live load acted horizontally and deflections were also measured horizontally, a plane in which the dead weight of the beam had no component. On removal of the superimposed load, recovery for no load can thus be observed. (Actually, the beam is secured in such a way that its center does not move even in a vertical plane due to the selected points of support.) The arrangement is shown in Fig. 1. The beams were loaded by a central point load, and deflections were observed for 5 weeks, after which they were unloaded and recoveries observed for at least 7 weeks. Four such cycles were repeated. For each W/C there were three beams that were loaded with 2, 4, and 8 kg (4.41, 8.82, and 17.64 lb), respectively. On loading and unloading, instantaneous deformations were also noted.

Test D—Axial compression

Electrical resistance strain gages (Philips type PR 9211, gage factor 2.06 ± 1.5 percent) were fixed on a cylindrical specimen, two in an axial sense and two in a circumferential sense, in four diametrically opposite points halfway along the height. For each cylinder there was a dummy cylinder of the same W/C and age. The cylinders were compressed in a 10-ton hydraulic loading machine. Loading was affected in 250 kg increments up to 2.0 tons, and readings were taken of axial and circumferential strains versus load. As it was desired to differentiate between immediate and delayed responses, this stage was carried out in a minimum time. After that the load was maintained (by means of the load-maintainer device) constantly at 2.0 tons for 48 hr, and strain gage readings were noted against time. The load was then removed and the strain gage readings were noted for both the instantaneous and the delayed responses, the latter up to 5 days.

RESULTS AND ANALYSIS

Fig. 2 is a typical result from Series A. It represents the load-deflection curves for instantaneous deformations up to fracture of beams with a W/C of 0.32, each curve being a mean of three or four beams, and the effect of age is shown. This confirms Bingham and Reiner's result that the material, right up to fracture, is obeying Hooke's law of proportionality. The accepted motion of a curvilinear relationship is therefore not correct when strictly instantaneous response is considered. Nonlinearity is due to delayed deformations which are included when the loading is carried out in the conventional manner.*

As these curves are the result of loadings rather than unloadings, there is no way of telling from this test whether the deformations are entirely reversible

*Nonlinearity may result from structural changes, such as internal cracking. This is not considered here since the present investigation was limited by the appearance of the first crack. This is the case in the bending of slender beams such as were used in the present investigation which fail completely on the appearance of this first crack.

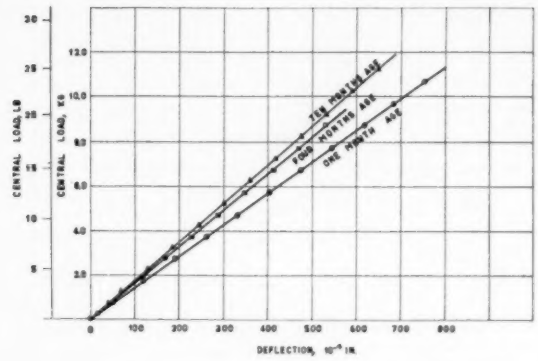

Fig. 2—Instantaneous response of hardened cement paste beams in flexure. Central point-loadings versus central deflections in simply supported beams for Series A. Each curve is a mean of three or four beams

or perhaps include permanent set* in which case the slopes of the curves are not true measures of Young's modulus. However, if a permanent set is present, it must be concluded that it, too, is proportional to stress.

Age increases the stiffness of the beams at a diminishing rate. From curves such as in Fig. 2 it appears that after 10 to 12 months elastic aging can be considered as completed.

Fig. 3 represents typical results of both Series B and C. The continuous curve is the time-dependent deformation, or creep, as obtained from Series B, for a load equivalent to 4.0 kg (8.82 lb) concentrated at the center. As mentioned before, the loadings in this test produced maximum bending moments increasing progressively by a factor of 2. The resulting curves, one of which is shown, indicated fair proportionality between rate of creep and bending moment. The cyclic curve is a result of Series C, and it represents the behavior of a beam under alternating load and no load, the load being equal to 4.0 kg (8.82 lb) in all four cycles. Other curves for other loads are not shown but all time-dependent deformations showed fair proportionality to load. Time-independent deformations showed good proportionality to load. The creep curve of Series B is a mean of three beams. The cyclic curve represents one beam. Only results for a W/C of 0.32 are illustrated, but the investigation indicated that the mobility,† as reflected from Series B, is affected by W/C, decreasing with it but at a decreasing rate.

The cyclic curve enabled the differentiation between the reversible and the nonreversible creeps, generally called primary and secondary creep, respectively.‡

It appears that for the load and for the time under load used, the secondary creep reaches a limiting value within the first 17 weeks. After that the material becomes purely elastic, although part of the elastic strain is not instantaneous but delayed, in that all deformations are then reversible. The fact that the

*Permanent set is here defined as the time-independent nonreversible deformation.

†Mobility, for the purpose of this paper, will be defined as the time-rate of deformation for unit load irrespective of whether the deformation is elastic, viscous, or of any other type.

‡To derive the curve for the secondary creep, an asymptotic value was assumed for the recovery of each cycle. The remaining was the residual deformation which included both secondary creep and permanent set. The latter, obtained from the difference between instantaneous deformations in loading and unloading of the respective cycle, was then subtracted from the residual deformation leaving a net value of secondary creep.

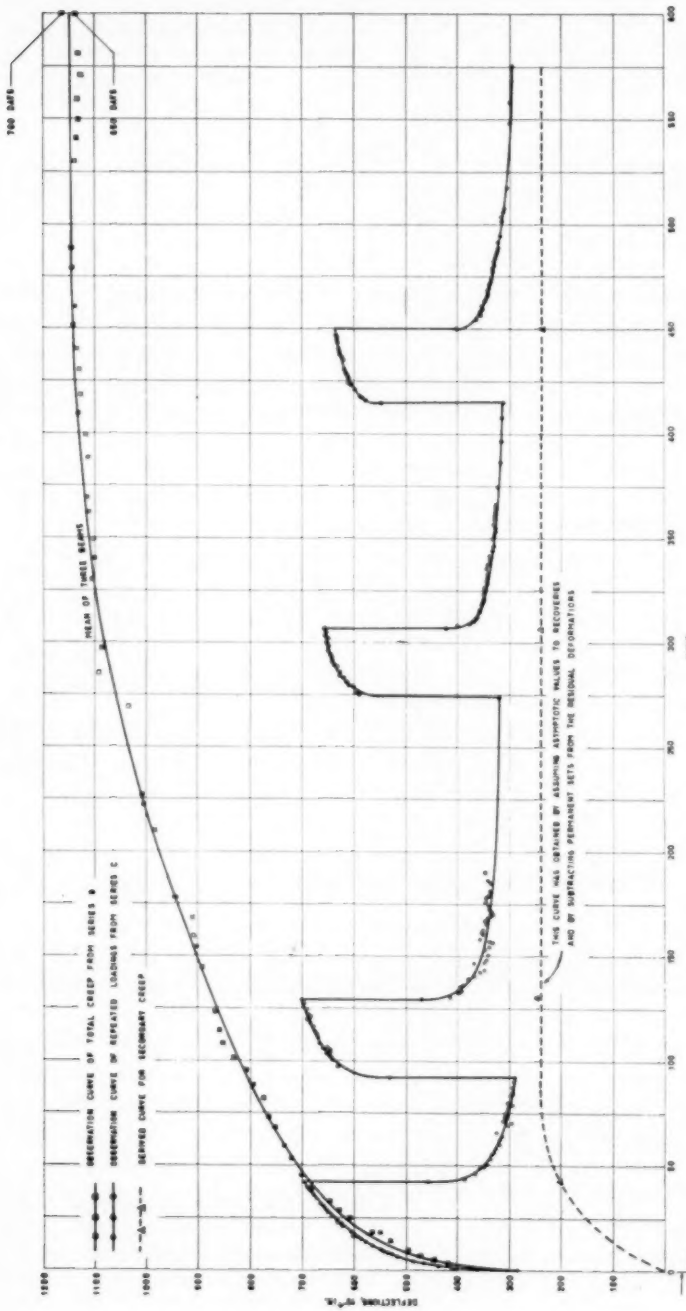
700 DAYS
650 DAYS
MEAN OF THREE BEAMS
OBSERVATION CURVE OF TOTAL CREEP FROM SERIES B
OBSERVATION CURVE OF REPEATED LOADINGS FROM SERIES C
DERIVED CURVE FOR SECONDARY CREEP
THIS CURVE HAS OBTAINED BY ASSUMING ASYMPTOTIC VALUES TO RECOVERIES
AND BY SUBTRACTING PERMANENT SETS FROM THE RESIDUAL DEFORMATIONS
DEFLECTIONS
DAYS

Fig. 3—Deflections at the center versus time of simply supported hardened cement paste beams in flexure for continuous and repeated loadings. Central point-load = 4.0 kg (beam dead load eliminated)

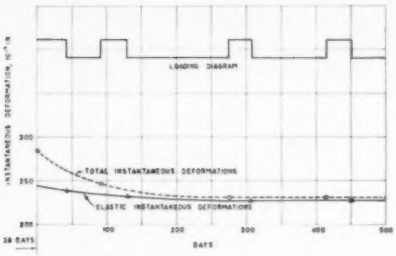

Fig. 4—Instantaneous deformations from a series of repeated loadings and unloadings for Series C. Central point-load = 4.0 kg

maximum deformation attained at the end of each loading phase falls slightly from cycle to cycle is probably due to aging which appears to influence the delayed elastic strains more than the instantaneous ones. If it was not for the aging, one would expect the total deformation of each cycle to exceed slightly that of the preceding cycle, the amount depending on the duration of preceding recovery, the reason being that each new loading commences from the still not completely recovered delayed elastic strain of the preceding cycle. The actual deformations, as seen from Fig. 3, exhibit this behavior when the effect of aging is taken into account.

Fig. 4 shows instantaneous deformations against time from Series C. In contradistinction to Series A, where differentiation between the elastic and permanent parts of these deformations could not be made, this could be done in Series C.* The difference between these two curves is the permanent set which is quite appreciable in the first cycle, still existent but smaller in the second cycle, and nonexistent after that.

Fig. 5 represents stress-strain curves of the loading (approximately time-independent) phase of Series D, which is the axial compression of cylinders. Only data for a W/C of 0.32 are shown. Both axial and circumferential curves are approximately straight lines and the ratio between the abscissas at any ordinate would be Poisson's ratio. At the low range of stress used, and with time-dependent deformations almost completely excluded, this linearity can now be understood.

*The instantaneous deformations measured in this series were plotted in the figure. As becomes evident when comparing these points with the loading diagram in the figure, the points representing "loadings" were connected to give one curve and the points representing "unloadings" connected to give another curve. The former is the "total instantaneous deformation" curve, and the latter the "elastic instantaneous deformation" curve.

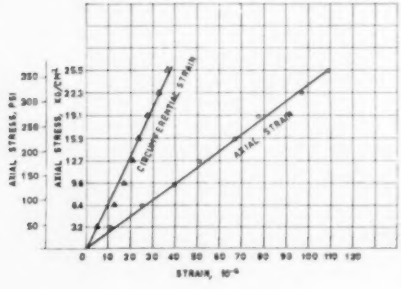

Fig. 5—Instantaneous response of hardened cement paste cylinders in compression. Axial stress versus axial and circumferential strains. Age = 1 month. Each point is a mean of two samples

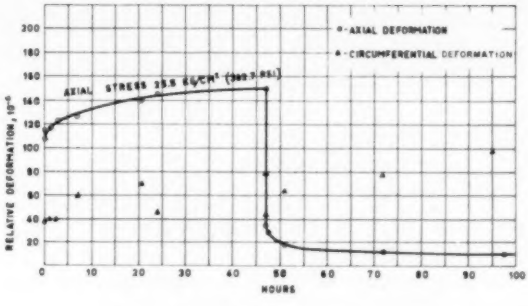

Fig. 6—Time-dependent response of hardened cement paste cylinders in compression. Age = 1 month. Each point is a mean of two samples

Fig. 6 shows the deformation versus time for the entire Series D for both axial and circumferential deformations. It appears that the former behaves as would be expected of a visco-elastic material and in agreement with the behavior in bending (Series C). The latter, however, shows an erratic response for the time-dependent phases indicating a noncontinuous process in lateral deformations.

DISCUSSION

Curves like those of Fig. 2 and 5 point to the fact, that whatever the rheological constitution of the material, it certainly possesses a free Hookean element. Young's modulus of this part of the rheological model is in the order of 250,000 kg per sq cm (230,000 ton per sq ft) and is only slightly dependent on age within 1 to 10 months.

This elastic deformation, however, does not constitute the entire instantaneous response, for there is also the permanent set as shown in Fig. 4. The interesting fact here is that this deformation is also present, if to a lesser extent, in the second application of the same load. The only explanation which can be suggested is that between the first unloading and the second loading a partial "healing" took place. This healing did not occur again after the second cycle presumably because the material had meanwhile aged. The results for pastes of other water-cement ratios, not recorded here, show a lesser healing effect with the decrease of water-cement ratio. At a W/C of 0.24, in fact, the instantaneous deformations became perfectly elastic immediately after the first cycle, a fact which incidentally supports the healing theory, as one would expect to connect the healing process with the amount of free water available.*

A rheological model is proposed for the permanent set as shown in Fig. 7. The Hookean spring is not continuous but contains, in series, elements which transmit force by friction. There is a great number of such elements with a spectrum of coefficients of friction. When a certain load is applied the spring is strained, but a permanent deformation also is formed through slippage of all the elements whose coefficient of friction is lower than the applied load.†

*Incidentally, there is nothing in the present investigation to show whether or not there will be a permanent set when unloading follows loading immediately, and not as happened in the present case with a delay of 5 weeks.

†In plastic materials these slippage elements are the representation of real slippage within the crystal lattice, whereas in brittle materials they represent the spreading of cracks up to their stabilization.

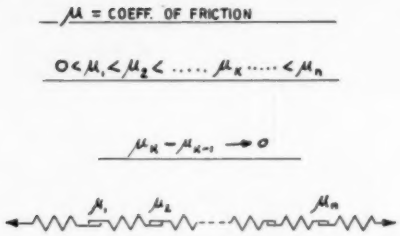

Fig. 7—A mechanical model for permanent set

The proportionality between permanent set and load is realized in the model by the number of slippage elements that operate for any one load.

For the creep behavior of the material two alternative theories may be suggested:

First theory

Assuming that the secondary (nonreversible) creep terminates after a relatively short time, as suggested by Fig. 3, one cannot look on it as a Newtonian flow whether linear or otherwise, as some authors did (e.g., Glanville,[3] Bingham and Reiner,[1] and Thomas[4]) and the material must be regarded as a solid not as a liquid. A rheological element that might represent this deformation should show some proportionality between load and deformation at any specific time after loading. An element meeting this requirement as well as the nonreversibility of the deformation is shown in Fig. 8. It is a Kelvin element made irreversible by means of a nonreturn valve in the piston (after Cowan[5]). On the basis of this model a relatively short time of retardation of about 2-4 weeks was calculated.

The primary creep, as proposed by Reiner,[6] can be represented by a Kelvin element. In our case the time of retardation was found to be about 20 weeks. A curve showing total creep, such as the continuous curve of Fig. 3 is thus a superposition of two exponential curves with unequal time factors as illustrated schematically in Fig. 9. The high curvature present at the beginning

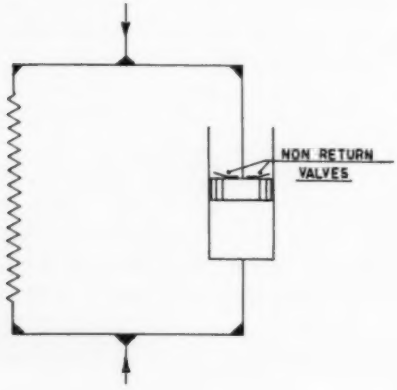

Fig. 8—A nonreversible Kelvin body

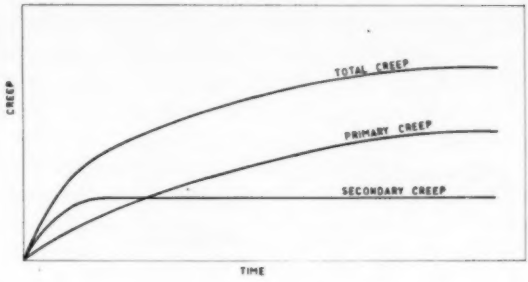

Fig. 9—Schematic curve of total creep as superposition of two deformations

of creep curves is accordingly the result of the secondary creep which, when subtracted from the total curve, will leave a much more uniform curve which is nearer a straight line on semilog paper.

The complete model representation of the material according to this theory is illustrated in Fig. 10a. The theory can be reconciled with accepted theories of internal structure of the material by interpreting the reversible delayed deformation as linked with flow of water within completely saturated zones, and the nonreversible creep as a reflection of the flow of water into undersaturated zones which is made nonreversible by virtue of capillary forces and the surface tension of water. Indeed, if the model is visualized as in Fig. 10b rather than 10a, the well-known dependence of secondary creep on submergence in water (not tested in the present work) is understood, and also the effect of shrinkage on creep.*

Second theory

The first theory was based on the assumption that the secondary creep ends several weeks after loading as seemed justified from Fig. 3. The evidence, however, is not conclusive as the duration of the loading phases was the same in all four cycles. If on loading for the fifth time, for example, the load would be left to act for, say, 10 weeks instead of 5, the total deformation would reach

*Theories presented here are based entirely on tests carried out by the author. The above references to the influence of submergence in water and to shrinkage on creep are therefore given with reservations. There is no doubt in the author's mind that these two factors influence creep but it will be necessary to perform tests similar to those presented here with samples submerged in water before the theory can be extended to cover such cases.

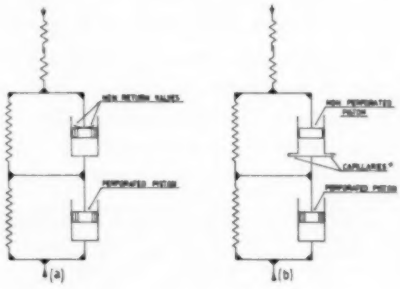

Fig. 10—Proposed models for the behavior of hardened cement paste according to the first theory

*Diameter of capillaries is such that capillary head is more than maximum possible force in the parallel spring

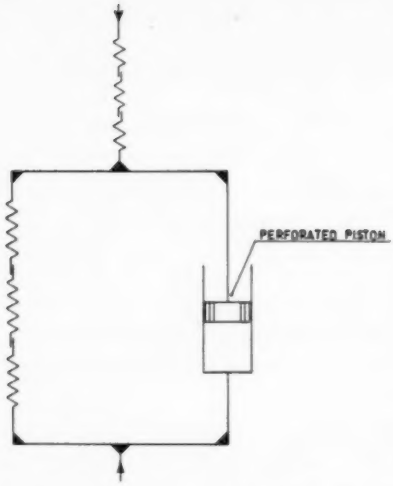

Fig. 11—Proposed model for the behavior of hardened cement paste according to the second theory

higher values than on previous loadings, and on recovering it may be discovered that the nonreversible deformation also increased. Since in this case the load would still be the same, the conclusion would have to be that the secondary creep depends also on deformation and not only on load, which would suggest that this creep is part of a deformation such as of a Kelvin body.

It will be recalled that in Series D the lateral creep was a discontinuous process in distinction to the axial creep. No blame can be placed on the strain gage technique (which is usually unsuitable for long term testing), since the axial gages behaved quite normally, and the conclusion must therefore be drawn that the reason lies with the formation of small *vertical* cracks *already under the low stresses used* (the highest stress reached was 25 kg per sq cm (356 psi). Jones[7] has already shown that only axial cracks form in an axially compressed cylinder, but the fact that such cracks appear under the lowest of stresses seems to be a new observation. Moreover, when loads were removed immediately after reaching 25 kg per sq cm (356 psi), all strain gages, including the circumferential ones, reverted back to their zero readings, which means that in these cases the cracks were not yet formed. These cracks would therefore be the result of what may be termed "static fatigue."

The static fatigue has been explained by the author[8] as the result of the gradual transference of load from the liquid to the solid phase in a Kelvin body. It therefore seems logical to assume that, if further testing along the lines described in the first paragraph of the second theory will show that indeed the secondary creep depends on the magnitude of the reversible deformation, the secondary creep is the macroscopic manifestation of the formation of minute cracks in the course of the deforming of a Kelvin body. Thus secondary creep is part, the nonreversible part, of a deformation of a nonperfect Kelvin body in the same manner as permanent set is the nonreversible part of the deformation of a nonperfect Hooke body. In fact we could name the secondary creep "delayed permanent set," and could represent the behavior of the material by means of a model as in Fig. 11.*

*It was previously stated that the scope of the present investigation is limited by the appearance of the first crack, yet here some of the mechanisms of deformations *below* this limit are explained by assuming some cracking. The discrepancy is one of degree; permanent set and the proposed delayed permanent set appear under the lowest of stresses, ultrasonic pulse technique will not reveal them, yet strain gages may. They are apparently caused by the formation of minute cracks. The cracks which cause deviation from linearity, and which lead ultimately to failure, are much larger, and are detectable by means of such techniques as the ultrasonic pulse method.

For the verification of this theory, further experimental work is necessary. It would also be of value to perform creep tests on beams with strain gages attached to the tension face of the beam. Erratic strains indicated by these gages in contrast with a continuous deflection as obtained by continuous readings of a dial gage in contact with the center of the beam, would provide a means of checking the theory. The erratic deformations will indicate a process of microscopic cracking and the continuous deflection of the center will then be explained as the macroscopic manifestation, or the aggregation, of these micro-deformations.

ACKNOWLEDGMENTS

This paper forms part of a thesis submitted for the DSc degree at the Israel Institute of Technology, Haifa. The research was sponsored in part by the Fohs and in part by the Ford Foundation. The problem was suggested by Prof. M. Reiner whose help is acknowledged with thanks. Thanks are also due Prof. R. Shalon, head, Building Research Station of the institute.

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Discussion of this paper should reach ACI headquarters in triplicate by Jan. 1, 1960, for publication in the June 1960 JOURNAL.

CONCRETE BRIEFS

Notes from Field and Office

Design of Anchor Bolts in Foundations 339

Design of Anchor Bolts in Foundations

By C. A. LEE*

It has seemed to the writer that some improvements could be made in the common practice in foundation bolting of heavy machinery. Let us consider for the first case machinery subject to heavy shock loads. We are reconciled to having occasional trouble with a bolt snapping off in the root of thread. When this happens, it is necessary to cut off the bolt below the top of the concrete and weld on a new threaded end. It seems that this sort of trouble could be eliminated.

Standard practice is to develop the strength of the bolt through bond. Stress is maximum at top of concrete but tapers off towards zero at foot of bolt. Unit strain is proportional to the stress at each point and cumulative in the length of bolt. The ratio of stress to total strain is of importance to absorb shock and would be decreased if bolt were under full stress all the way down. Any decrease in ratio is favorable with shock loads. In other words: shock loads would be better handled if bond were destroyed and end anchorage designed accordingly.†

We can go further—assume that the bolt is 4 in. in diameter with 6 ft embedment. A 4 in. threaded stub could be welded to a 3-in. shank, with an anchor plate at the bottom. Then highest stress would come in the 3-in. shank rather than at root of the 4-in. thread. Strain will be distributed accordingly. This principle has been successfully used in connection with heavy grinding mills. It is worth serious consideration whenever shock load is a problem (see Fig. 1).

Fig. 1—Anchor bolt with shank smaller than thread; greased to destroy bond

*Member American Concrete Institute, Design Engineer, U. S. Steel Corp., Chicago, Ill.

†Strain is in proportion to stress for the concrete as well as for the bolt. The concrete is actually compressed as bolt is stretched. The bolted-down machine base may be considered as floating between two stiff springs: the steel bolt and the concrete foundation. Deformation is divided between the two. Under shock load, there is a cyclic redistribution of the deformations. A lower ratio of stress to total strain means now that stress does not increase so much with increase in strain for upper half of cycle. There is not so much stress at root of thread.

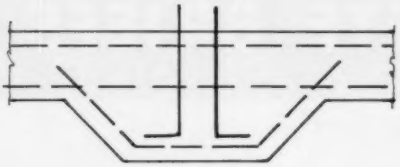

Fig. 2—Extra slab thickness required to give embedment necessary for specified bond

A second problem is that in which shock load is not seriously involved. Consider a hundred bolt-cluster distributed over a 5000 sq ft mat 21 in. thick. Assume that mat is reinforced top and bottom and that there is no question as to strength of the mat in properly distributing the various concentrated loads. If the bolts are 1½ in. in diameter many specifications require 45 in. embedment below the sleeve; and extra depth of slab will be required for each group. Extra layout work is required and a lot of pick-and-shovel work. Probably extra bent-bar reinforcement will be thought necessary as shown in Fig. 2.

Suppose bond is neglected and the bolt is designed on a basis of shear as a measure of diagonal tension. A substantial anchor plate would be required but could be based on a depth of 17 in. embedment and extra concrete depth would not be required (Fig. 3). Obviously there would be a substantial saving.

In a way, it is interesting that it should be thought necessary to thicken footings simply to provide required for anchorage of bolts through bond. We drill holes in existing concrete and set rag-bolts 10 or 12 diameters deep. We set bolts with expansion anchors at still less depth with reasonably satisfactory results.*

The Problems and Practices section is an open forum on everyday topics. JOURNAL readers are invited to contribute their solutions to problems that occur in the field or office—whether common or unusual.

Perhaps your solution is a better way of handling some everyday problem that others are looking for. Give your fellow ACI members the benefit of your experience on these problems.

Items appearing in this section can vary from 100 to 1000 words. It is an ideal place for those practical items generally considered too short to be submitted for publication, but containing material of potential importance to a great many.

A moderate amount of local anchorage or reinforcement would permit using shallow bolts without spalling of concrete if there were any point in doing so. In other words, if a slab were designed with sufficient thickness for the loading within a reasonable margin, it should be possible to set foundation bolts in that slab. A suggestion is shown in Fig. 4.

These remarks are intended to show that bond is not the only means to develop strength of a foundation bolt. We should be willing to use a different basis to meet special conditions. Of course, in the ordinary case there is no objection to making full use of bond stress.

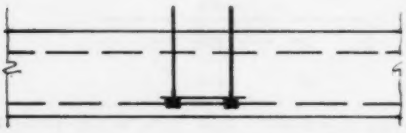

Fig. 3—Anchor bolt designed on basis of diagonal tension. Large anchor plate required but only 17 in. embedment

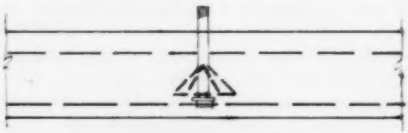

Fig. 4—"Shear-head" style reinforcing for anchorage using wire mesh rolled into a cone

*Adams, Robert F., "Some Factors Which Influence the Strength of Bolt Anchors in Concrete," ACI JOURNAL, V. 27, No. 4, Oct. 1955 (*Proceedings* V. 55), pp. 131-138.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

PRESTRESSED PAVEMENT BIBLIOGRAPHY

By SHU-T'IEN LI*

FOREWORD

Sections of prestressed pavement have come into existence only after World War II and the technique of its design and construction is still undergoing development. Any attempt to codify the immature practice at this early stage would not seem as urgent as a free-world-wide bibliography of the relevant literature.

The bibliography presented here represents the author's gradual accumulation since the first prestressed pavement was constructed at the Paris international airport at Orly, France, by Freyssinet, who has laid down the foundation of prestressed concrete in all its applications.

While prestressed pavement was being introduced to the Orly airfield by Freyssinet, the author was inaugurating an advanced course in airport engineering at the National Peiyang University, China. The course was given only to advanced civil engineering students. It was conducted in systematic lectures without textbooks. Within a general framework, the contents of the syllabus were revised from time to time to incorporate the most up-to-date theory and practice. It was for this course that this bibliography was accumulated, as a part among other things, and kept in chronological order to reveal the advancement of this branch of knowledge.

This bibliography covers papers, articles, and reports published in the technical press and society proceedings of 13 countries: Australia, Belgium, Canada, China (Formosa), Denmark, France, Germany (West), India, Italy, Japan, Spain, United Kingdom, and United States from November, 1946 to February 1959.

The ably presented, well written, and thoroughly digested report of Subcommittee VI, ACI Committee 325, on the world view of the status of prestressed pavement truly represents the first concerted effort on the subject in the United States and will undoubtedly provoke much interest. This bibliography is submitted to complement the subcommittee report.

It is interesting to note that no actual construction or test section, laid anywhere since the introduction at Orly, has surpassed the dimensions of

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that project. There the pavement slab was not only prestressed but also precast. The strip is 1368 ft long, 195 ft wide, consisting of precast slabs, each a meter square and 6½ in. thick. The installation uses 45-deg diagonal joints making only transversal prestressing necessary. Anchorage of prestressing steel is secured by heavy abutments. Freyssinet claims that a 6½-in. prestressed slab is capable of supporting loads equal to those of a 24-in. conventional concrete runway. Further, he claims his runway can support aircraft three times the weight of any yet built. This claim was confirmed when extensive load tests were made on a test slab at the Orly Aerodrome during 1946 and supported a load nearly 10 times greater than that which could be expected on the same subgrade when using a conventional concrete pavement.

No further summary of other experimental installations will be attempted in this brief foreword. The readers who are interested in prestressed pavement will get first-hand, original exposition from the literature cited in the bibliography. The bibliography has been confined to prestressed pavement. To insure successful installation of a prestressed pavement, all the latest applicable developments associated with rigid pavements must, of course, be taken into consideration during design and construction.

Among the points to be considered are:

1. Continuous type pavement without prestressing steel, or individual type with prestressing steel
2. Optimum method of placing the steel; diagonal or longitudinal with transverse steel
3. Optimum position of steel
4. Optimum amount of steel
5. Prestressing with high-tensile steel wire-cable and cone, or by bar and accessory
6. Post-tensioning, or jacking against abutments
7. How to remove the difficulty of pretensioning
8. Optimum method for anchoring the steel
9. Kind of steel sheaths, pipes, and saddle supports
10. Optimum thickness of slab
11. Precast, or cast in-situ
12. Pattern of slab; rectangular, square, or triangular
13. Criteria for creep, temperature, moisture, and frictional restraint stresses
14. Criteria for design value of residual prestress for different classes of loading
15. Optimum spacing of construction joints; longitudinal and transverse
16. Unrestrained or restrained expansion
17. Optimum spacing of expansion joints
18. Appropriate size of expansion joints
19. Kind of construction for expansion joints
20. Whether special expansion joints are needed at ends of prestressed slabs
21. How to construct horizontal curves with superelevation
22. How to construct vertical curves
23. How to construct overlay type prestressed pavement
24. Quality of aggregates
25. Types of cements; possibilities of expansive cements
26. Optimum water-cement ratio
27. Desired strength of concrete
28. How to strengthen the concrete where the jack applies a heavy load to prevent concrete from spalling
29. How to prevent wedges from flying out
30. Relative merits of prestressed pavement on poor, compressible subgrade, and on good subgrade
31. Optimum preparation of poor, compressible subgrade
32. Optimum preparation of good subgrade
33. Optimum method of reducing subgrade friction

34. How to insure riding quality
35. How to provide repairs for utilities
36. Provision for load tests
37. Preplanned schedule for taking performance records
38. Optimum development of construction equipment
39. How to reduce unit costs
40. How to organize joint research to benefit all concerned

There can be no doubt of inherent economy of prestressed pavement; its physical advantages insure such. The ever increasing wheel loads and traffic frequency over highway and airport pavements further accentuate this fact in favor of the new composite material versus conventional reinforced concrete. As more actual bids on prestressed pavements are called for, contractors will rapidly increase in experience and become better provided with both equipment and skilled labor thereby decreasing the construction cost; this has already proved to be so in other applications of prestressed concrete. The present barrier does not rest in the prestressed pavement itself but rather in the reluctancy of engineers to use it. The bibliography reveals convincingly that the prestressed pavement has been used in countries subjected to more economic pressure.

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Bridges

Merdeka Bridge, Singapore: Investigation, design, and construction

REGINALD JOHN HOLLIS-BEE, *Proceedings*, Institution of Civil Engineers (London), V. 11 (Session 1957-58), Oct. 1958, pp. 135-168

Reviewed by ARON L. MIRSKY

Describes an interesting and ingenious solution to the problem posed by a swampy site with extremely poor foundation conditions. For minimum weight and maximum economy, the twenty-five 80 ft spans were constructed on a "bund" in which channels were later cut; precasting and prestressing were fully utilized. The piers are hollow, of precast A-frames with cast-in-place sheathing walls, and founded on concrete piles and caissons. The main deck beams were precast and post-tensioned, and the deck system was precast and transversely post-tensioned. Design, construction, and testing (including testing of a beam to destruction, investigation of the lateral distribution of live loads on a finished span, and testing and control of concrete) are discussed.

"Wafer Slab" bridge decks

P. W. ABELES, *The Engineer* (London), V. 205, No. 5341, June 6, 1958, pp. 842-843

Reviewed by ARON L. MIRSKY

Article (abstracted from a paper presented to the Third Congress of Fédération Internationale de la Précontrainte, Berlin, May 5-12, 1958) discusses a shallow bridge deck design developed by the Eastern Region of British Railways for use on electrified lines where construction depths must be held to a minimum. The deck units are cast and prestressed in two stages, the first stage consisting of precast inverted T-beams and the second stage of concrete placed when the beams are so supported that they are free of dead-load stresses, and utilize a more highly stressed skin; tensile stresses are permitted in the concrete. Reinforcement at the joints of adjacent units provides satisfactory transverse distribution.

Construction Techniques

Rail-riding buggy slashes paving costs

ALFRED PEREIRA, *Construction Equipment*, V. 20, No. 1, July 1959, pp. 31-35

One structure on the Cross Bronx Expressway is a 1700 ft long viaduct with two 350 ft approaches and calls for a monolithic placement of 11 three-lane-wide deck panels. The width of the slab and the height above ground (about 40 ft) make it almost impossible for a crane to place the concrete directly on the deck. The contractor developed a mechanical dumper for spreading the concrete. It consisted of a 2-yd hopper mounted on the outside end of the bridge and a buggy traveling on a bridge to place concrete at any spot. This rig cuts 3 hr off paving time and saves about 80 man hr a day, cutting paving costs by about $300 a day. The article also discusses the new deck design, the transverse screed finisher, and aids for final finishing of the concrete.

Design

Design of arch ribs for reinforced concrete roofs

J. S. TERRINGTON, Concrete Publications Ltd., London, Reprint, 1957, 28 pp., $6

Title changed from *Design of Arch Ribs* (1937 edition) because of introduction of the thin slab vault or shell type of arch rib with which this book does not deal. Gives the method of calculating bending moments, thrusts, and shears at any point in the arch rib type of roof. Discusses method of analysis, and vertical and horizontal loading, and then goes into examples for following conditions and loadings: (1) free end condition—vertical loading; (2) fixed end condition—vertical loading; (3) fixed ends—horizontal wind load; (4) free end condition—horizontal wind load. Tables give values for bending moments, normal thrusts, and shears at various sections on the arch.

Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

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On judging the danger of earth shocks to structures (Über die Beurteilung der Gefährlichkeit von Erschütterungen bei Bauten)

J. Geiger, *Der Bauingenieur* (Berlin), V. 34, No. 1, Jan. 1959, pp. 19-23

Reviewed by Aron L. Mirsky

Author demonstrates that the characteristics of a structure are as important as the magnitude of the earthquake vibrations to which it may be subjected, sets up and solves the differential equation for the vibration of a simple system, and shows that it is important to measure the vibrations of the component parts and the upper portions of a structure. Apparatus for measuring vibrations of small amplitude was described in author's previous paper (*Der Bauingenieur*, V. 30, No. 6, June 1955, pp. 210-217; "Current Reviews," ACI Journal, Nov. 1956, p. 560).

Damaging earthquake and blast vibrations

Frank Neumann, *Trend in Engineering at the University of Washington*, V. 10, No. 1, Jan. 1958, pp. 4-10, 24-28

Oscillator responses to earthquake motions—a new approach to the lateral force problem

Frank Neumann, *Trend in Engineering at the University of Washington*, V. 10, No. 4, Oct. 1958, pp. 4-10, 24-25

Reviewed by Aron L. Mirsky

In the words of the author, "These two papers complement each other and explain why the seismic force that throws over a bookcase is not the same force that causes structural deformation in the building housing the bookcase."

First paper concerns intensities and damage related to absolute accelerations. It discusses displacement and acceleration records, the two types of quake or blast damage (that due to acceleration or "force" and that due to shear or flexural deformation), the response of structures to different frequencies, etc., and reconciles the conclusions of Crandell, Thoenen and Windes, and the author in the form of a composite diagram relating ground acceleration, ground frequencies and periods, displacement, number of stories in buildings, and effects.

Second paper concerns deformations due to relative motion between the center of oscillation of a structure and the moving ground. It discusses earthquake spectra, the author's quantitative earthquake intensity grid and its use as a tool for measuring spectrum intensities, and the determination of the maximum lateral force; and makes recommendations for further studies.

On the solution of rigid frames by the column analogy

Thomas D. Y. Fok, and Tung Au, *Proceedings*, ASCE, V. 85, No. ST1, Jan. 1959, pp. 103-112

The paper presents rigorous proofs of the applications of column analogy to the analysis of single-span rigid frames whose supports may be fixed or hinged.

A new approach to the calculation of bar and bond stresses in reinforced concrete (Ein neuer Ansatz zur Berechnung der Draht- und Haftspannungen im Stahlbeton)

Hans Bufler, *Der Bauingenieur* (Berlin), V. 33, No. 10, Oct. 1958, pp. 382-388

Reviewed by Aron L. Mirsky

Assuming only that Hooke's Law holds and that equilibrium is satisfied, author sets up the general differential equation which he then solves for several major cases (temperature change, creep, shrinkage, prestressing, etc.) by considering the pertinent boundary conditions. The reinforcing steel is assumed to be fully bonded, and the concrete is taken to be in the form of a prismatic body (more exactly, a cylindrical specimen) with axially-placed reinforcement; no bending is admitted. Results are compared with photoelastic experiments (models of Araldit with glass and duralumin fibers as reinforcement, as used by de Vasconcelos) and with empirical formulas which are shown to give results in good agreement; discrepancies are analyzed.

Rectangular flat slabs supported on columns (in Swedish)

Henrik Nylander, *Nordisk Betong* (Stockholm), V. 3, No. 2, 1959, pp. 155-176

Reviewed by Margaret Corbin

Presents an analytical method for determining the moments in a two-way reinforced rectangular flat slab supported along the edges and carried on a column in an interior panel. Determination of moments is based

on the yield line theory (Johansen). This theory yields a higher limit of load carrying capacity than that obtained by means of the plastic theory, which disregards the membrane effect. Thus a selection must be made among various conceivable types of yield lines to arrive at the type resulting in lowest load-carrying capacity. The requirement is that the type of yield lines be relatively simple. Results of investigations by the author in an earlier publication are used to determine optimum moment distribution and a comparison with the elastic theory is made. Application of the design method described is illustrated by examples.

Design of pyramid roofs

J. S. Terrington, Concrete Publications Ltd., London, reprinted 1957, 22 pp., $1

Discusses types of pyramid roofs and their stability under vertical loading. Develops formulas for three different cases of connection and support: Case 1, a structure having no connection between the adjacent planes at the ridges and supported vertically and horizontally along the lower edges; Case 2, a structure connected at the ridges and supported vertically only along the lower edges; Case 3, a structure connected at the ridges but supported vertically at the lower corners only. Under each case, the following effects are discussed: (a) the primary thrust down the triangular surfaces; (b) the secondary thrust at right angles to it, that is, parallel to the bottom edges; (c) the weight of each panel acting normal to the plane of the surface, causing bending as in the triangular slabs supported on three edges. After developing the formulas, examples are worked for the various conditions.

Wind effects on roofs

Building Research Station Digest (England), No. 122, May 1959, 8 pp.

Discusses the characteristics of wind and factors affecting its speed, such as: geographical variability, topographical variability, and effect of buildings on wind flow. Under pressures due to wind gives basic wind pressure formula and design wind pressure based on basic wind pressure. Describes internal pressures, effect of pitch of roof, local pressures, direction of wind. Other factors are considered—strength and stability, overhangs, mono-pitch roofs, and the special applications of lightweight house roofs of low pitch or other special roofs. Charts show recorded wind velocities for 9 hr, maximum gust speeds throughout England, pressures as affected by height of building, pitch of roof, and type of roof.

Economical design of reinforced concrete structures (in Swedish)

Arne Rinkert, *Nordisk Betong* (Stockholm), V. 3, No. 1, 1959, pp. 67-76

Reviewed by Margaret Corbin

A relatively low percentage of reinforcement can be most economical in structures which are submitted to moments due to live loads alone. The grade of concrete should also be taken into account. If the structure is acted upon by moments due to live loads alone, the concrete should be as low-grade as possible. On the other hand, if the structure is also subjected to moments due to dead load, the concrete should be as high-grade as possible.

Design of domes

J. S. Terrington, Concrete Publications Ltd., London, reprinted 1957, 31 pp., $1

Develops and applies the theory for the design of dome shell roofs. Discusses the general conditions for stability and calculation of stresses due to uniform load and concentrated load at the crown. Then, applies this in an example of the design of a hemispherical dome shell. Develops formulas for thin spherical domes of varying thickness and works example of application. Describes graphical method and then checks results of analyses of previous examples. Applies theories to flat dome roofs, pile foundations, and conical dome.

Analysis of a cylindrical shell roof with edge beams

F. A. Gerard, *Transactions*, Engineering Institute of Canada (Montreal), V. 2, No. 4, Dec. 1958, pp. 168-174

Author's thesis for Ph.D. (London, 1956), "The Analysis of Cylindrical Shell Roof Structures" deals with a radically different and original method for analyzing cylindrical shell roof structures. This article is taken from Chapter V of the thesis with the subject

matter condensed and rearranged so that the discussion would meet the requirements of those whose prime interest is in method application. Solution is outlined and illustrated by worked examples, theory is discussed in Appendix A, and a further worked example involving prestressing of edge beams is discussed in Appendix B. Works several examples and compares stresses or forces at different points to results by several other methods.

Shear strength of reinforced concrete beams

SIDNEY A. GURALNICK, *Proceedings*, ASCE, V. 85, No. ST1, Jan. 1959, pp. 1-42

"Shear failure" of concrete beams is recognized as the failure under combined compression and shear of the concrete above a diagonal crack. Rational interaction equations for the failure of plain concrete under this stress combination are developed on the basis of Mohr's failure theory. For the equations to be applicable, the strength of the particular concrete in direct compression as well as in direct tension must be known. The latter is determined satisfactorily from the modulus of rupture. The equations are used to develop a rational theory of shear strength of beams without web reinforcement. A tentative extension is made to web-reinforced beams. Numerous tests are cited which show satisfactory agreement. (The paper is a condensation of a doctoral dissertation prepared at Cornell University under the direction of George Winter.

Materials

Use of sodium alignate for improving concrete mixes

D. DALEV, L. LIDZHI, K. BAEV, and D. DANCHEV, *Comptes Rendus*, Academie Bulgare Science (Bulgaria), V. 10, No. 2, 1957, pp. 117-9. Reviewed in *Chemical Abstracts*, V. 52, No. 4, 1958, p. 3299; *Building Science Abstracts*, V. 31, No. 7, July 1958, p. 201

HIGHWAY RESEARCH ABSTRACTS Mar. 1959

Sodium alignate made from a Black Sea seaweed improves the consistence, compaction, and frost resistance of concrete. With a water-cement ratio of 0.62, the 28-day strength can be increased by about 8 percent by addition of 0.01 to 0.05 percent of the additive. The stability of the dispersion of the cement particles is so high that the concrete is not segregated by vibration during transport.

Initial grinding of blast furnace cement components (De la prémouture des composants du ciment de haut fourneau)

L. BLONDIAU, *Revue des Matériaux* (Paris), No. 513, June 1958, pp. 153-158; No. 514-515, July-Aug. 1958, pp. 204-207; No. 516, Sept. 1958, pp. 243-255

Reviewed by PHILLIP L. MELVILLE

A study of separate grinding of slag and clinker used for the production of blast furnace cement. To obtain blast furnace slag cement having equal fineness to portland cement requires 20 percent more power, and to be equal in strength it requires 80 percent more power. The question is studied from different angles: higher strength, effect of long-time storage, and power consumption.

Results indicate that separate grinding does not improve cement quality while using more power but that it does somewhat reduce deterioration after long storage periods.

A new system of notation for mortars proposed by the Scandinavian mortar committee (in Swedish)

VITOLD SARETOK, *Nordisk Betong* (Stockholm), V. 3, No. 2, 1959, pp. 145-154

Reviewed by MARGARET CORBIN

All figures are given by weight. Examples: a lime mortar, now denoted 1:4 by volume, should be designated K 100/800. A cement mortar, 1:4 by volume, is denoted C 100/400. A lime-cement mortar, 2:1:12 by volume, contains equal quantities of lime and cement (by weight), and is designated KC 50/50/600. The new system is intended to replace the traditional systems, which are based on proportions by volume.

Arizona volcanic cinder concrete: A comparative study

A. W. ROSS and K. K. KIENOW, *Bulletin* No. 9, Civil Engineering Series No. 4, Engineering Experiment Station, University of Arizona, May 1959, 63 pp.

In Arizona, deposits of volcanic cinders occur in many areas where there are no naturally occurring sources of sound, dense aggregates. Concrete construction in the areas having a dearth of aggregates has been

expensive because of aggregate hauling costs. This study finds that volcanic cinder aggregates of the types tested can produce concretes having compressive strength suitable for structural use with a reasonable degree of economy. These concretes have somewhat inferior finishing qualities, but for ordinary structural use, the workability is acceptable. Describes materials, testing methods and procedures, results and conclusions, and design criteria and procedure. Appendix contains plots of test results, tabulation of material properties, mix proportions, and strengths.

Petrographic investigation of quaternary Danish gravel deposits (Petrografisk undersøgelse af Danske kvartaere grusaflejringer)

B. SØNDERGAARD, *Progress Report* E 1, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and Academy of Technical Sciences, Copenhagen, 1959, 74 pp. 12 D.kr

AUTHOR'S SUMMARY

The petrographic investigation comprises 136 samples of quaternary Danish gravel deposits (glacial deposits, beach deposits, and materials pumped up from the sea bottom). The material was subjected to sieving in fractions between ⅛ and 64 mm, and the quantity of the different rocks determined. The result of the analyses is given *in extenso* in tables at the back of the book.

High-alumina cements

T. D. ROBSON, *Architectural Science Review* (Sydney), V. 2, No. 1, Mar. 1959, pp. 49-56

High-alumina cements are now made in England, France, Spain, Yugoslavia, Germany, Czechoslovakia, Italy, Japan, United States, and Russia. This article discusses the history of the cement, including manufacture, and goes into the constitution, giving chemical composition for the trade names in various countries. Discusses the action in hydration and physical properties, especially setting time, compressive and other strengths, and miscellaneous properties. Then, discusses structural uses, especially where chemical resistance properties or refractory properties are desired. Concludes with warnings, especially that (1) concrete with high-alumina cements must be water cured during the first 24 hr; (2) calcium chloride should not be used; (3) most waterproofing agents are deleterious in high-alumina cement mixes; (4) sea water should not be used for gaging; (5) the use of unduly rich mixes for structural high-alumina cement concrete is rarely desirable or necessary. One table compares the development of strength of portland cement, high-alumina cement, rapid hardening portland cement (high-early-strength) with and without calcium chloride.

Mineral aggregates: 1958 revision

Bibliography No. 23, Highway Research Board, 1958, 112 pp.

This 1958 revision includes and brings up to date a prior annotated bibliography on mineral aggregates (Highway Research Board *Bibliography* No. 6) published in 1949. It comprises a chronologically arranged annotated bibliography on mineral aggregates including everything available up through 1956. Complete indexing both by subject and by author makes location of a specific reference easy. Also included is a supplementary listing of references on distribution of mineral aggregates arranged by states as included in the original 1949 work.

Pavements

Sealers for joints and cracks in concrete pavements

Bulletin No. 211, Highway Research Board, Mar. 1959, $0.50

Laboratory tests of sealers for sawed joints

WILLIAM H. KUENNING, pp. 1-12

Reports on the performance of sealing materials for sawed joints being studied at the Portland Cement Association Laboratories during 1955-57. This is a continuing program of extensibility tests on such sealers, both conventional and in the developmental stage, in which the joints are opened and closed at speeds of the order of 1/32 in. per hr. Shows that extensibility is not strictly an inherent property of the material but is highly dependent on the shape factor. Initial joint widths must be designed in relation to the joint sealer material, joint spacing, and climate. Discusses the materials, tests, types of failure, costs of sealers, and concludes with several recommendations.

Comparative testing of joint sealers in sixteen laboratories

EGONS TONS, pp. 13-22

Although the same procedure and specifications are followed for the testing of joint sealers, the results may be different for different laboratories. To find out how different these results could be, 16 laboratories were asked to participate in the evaluation of bond-ductility, flow, and penetration tests for joint sealing compounds. In spite of the careful planning and control the test results vary considerably between the participants. This paper discusses the standardization of the samples and the testing procedures and equipment and conditions. The conclusions discuss the results of the tests from the various laboratories and make recommendations for improving test procedures.

Road pavement thickness design methods in Australia

V. P. O'GRADY, *Journal*, Institution of Engineers, Australia (Sydney), V. 31, No. 3, Mar. 1959, pp. 51-55
AUTHOR'S SUMMARY

Gives a brief description of the various methods used in Australia for determining desirable road pavement thicknesses and tests conducted and recorded over the past 12 years by member Authorities of the Conference of State Road Authorities of Australia.

Concrete paving practice: Anything but static

H. A. RADZIKOWSKI, *Roads and Streets*, V. 102, No. 5, May 1959, pp. 122-134

Discusses progress made in concrete paving methods and also some future expectations. Improvements in compacting and grading equipment, aggregate upgrading equipment, batching equipment, improved conventional pavers, and other equipment, are described along with new type mixers, slip form pavers, positive type tampers, and concrete saws. Paver mixing time, use of ready-mixed paving concrete and additives are described as well as some of the concrete paving research, end result specifications, and new testing equipment. Cites several future needs in development and improvement.

Precast Concrete

Concrete trees carry forestry lab roof at Yale

Engineering News-Record, V. 62, No. 19, May 14, 1959, pp. 36-37

A report on the construction of a one-story, 106 x 169-ft School of Forestry building in New Haven, Conn. This building employs precast concrete columns with split, branching tops, that resemble the letter Y. The roof panels, 3 ft wide and 21-27 ft long, have concave convex rounded ends which give a sine-wave-shaped line running the length of the building. Formwork for this job was intricate due to the unusual shape of columns and roof panels. Wood and plaster molds were used for the roof panels and required careful maintenance as casting progressed.

Prestressed Concrete

Prestressed concrete: theoretical and experimental study; Vol. 2, Hyperstatic structures (Béton précontraint: étude théorique et expérimentale; Tome II, Constructions hyperstatiques)

Y. GUYON, Éditions Eyrolles, Paris (for the Institut Technique du Bâtiment et des Travaux Publics, Paris), 1958, 818 pp., 9300 fr. (about $19)
Reviewed by ARON L. MIRSKY

When the first volume of this work was reviewed in the ACI JOURNAL,* it was characterized as truly remarkable. The same term can in all justice be applied to the second volume as well.

But there is another term which comes to mind, and which is perhaps even more descriptive: labor of love. To conceive, research, correlate, and produce such a monumental work indicates an author who is not only convinced of the value of his subject, but is in love with it.

Merely to attempt to list the subject headings would require considerable space, and would not even begin to indicate the full extent of the contents. The volume is treated (in all except pagination) as a con-

*V. 24, No. 7, Mar. 1952 (*Proceedings* V. 49), pp. 607-608. Also the English translation was reviewed in ACI JOURNAL, V. 26, No. 8, Apr. 1954 (*Proceedings* V. 51), p. 693.

tinuation of the first volume, which was divided into three parts. In this volume, Part 4 covers the elastic analysis of indeterminate (hyperstatic) systems (continuous beams, rigid frames and arches, building frames, etc.) of prestressed construction, and includes a lengthy discussion of parasitic (secondary) reactions and of methods of draping the prestressing cables to reduce or eliminate these unwanted secondary reactions. Included also are a chapter on joints, particularly temporary joints and construction joints (these last being of especial importance if precast construction is used), and a chapter on the relative economics of isostatic and hyperstatic construction with prestressing.

Part 5, on nonelastic behavior, reports tests on various hyperstatic prestressed systems (including two chapters on continuous slabs), plasticity (the moment-curvature law, plastic rotations, limitations), and a final chapter on ultimate design and its application to prestressed indeterminate structures.

This work has evidently been intended as a textbook. As such it is marked by clear and full exposition, numerous sketches, descriptions of various recent structures (both French and non-French), and many fully-worked examples. There is little doubt that anyone who masters its contents will have gained considerable knowledge of the field (at least as of the time the book was written!). It is thus doubly unfortunate, from the viewpoint of the serious student or the practicing engineer, that the book has no index; not even the fairly detailed table of contents, at the end of the book, can fully take the place of that.

A selected bibliography, to supplement the rather sparingly used citations scattered through the book, would also have been a welcome and useful addition.

Nevertheless, this book is a real accomplishment. It—or an English translation, which reviewer hopes will not be long in forthcoming—belongs in the hands of everyone seriously interested in indeterminate prestressed structures, and on the shelves of every library catering to structural (not handbook) engineers.

Losses due to friction in curvilinear post-stressing cables (in Polish)

R. Kowalczyk, *Inzynieria i Budownictwo* (Warsaw), No. 1, 1958, pp. 6-10

Polish Technical Abstracts No. 3 (31), 1958

A mathematical analysis of losses in post-stressing forces due to friction. Determination of friction coefficients in cables is made and its relation to the above losses is discussed. Effect of cable rigidity on these losses is also given.

Research on anchoring cables in steel sheaths in post-stressed concrete construction (in Polish)

M. Wolff, *Inzynieria i Budownictwo* (Warsaw), No. 6, 1958, pp. 210-213

Polish Technical Abstracts No. 4 (32), 1958

Tests concern the anchorage of spiral cables. The cables consists of 37–5-mm diameter wires. The cables are first given a zinc bath and then secured in special sheaths made of steel cylinders provided with turned conical openings. The most effective wedge action was obtained with an obtuse angle of cone inclination. Cables should be loaded four or five times during post-stressing with a load 5 to 10 percent greater than what is actually required. Twisted cables proved to be more effective in post-stressing than cables consisting of parallel wires.

Properties of Concrete

Method of taking impressions for examination of porous surfaces (in Swedish)

Erik Hogberg, *Nordisk Betong* (Stockholm), V. 3, No. 2, 1959, pp. 139-144

Reviewed by Margaret Corbin

This article describes a method of taking impressions which can be used to facilitate the study of frost resistance of concrete produced by air-entraining agents. Accurate impressions of brightly polished concrete or mortar surfaces can be taken by means of the same impression compound employed in dental practice. The imprints of the air pores on the impression are easier to observe and to measure in microscopic examination than the actual air pores on the original surface.

Some investigations into the elastic properties of concrete

R. G. ROBERTSON and D. C. ROBERTSON, *Transactions*, South African Institution of Civil Engineers (Johannesburg), V. 6, No. 9, Sept. 1956, pp. 277-282
Reviewed by D. G. NORMAN

Tests on about 100 specimens gave figures for cube strength, Young's modulus (in tension and in compression), column strength (4 x 4 x 24 in.), tensile strength, and modulus of rupture. Stress-strain curves for tension and for compression are also given.

Resistance of concrete and mortar to chemical attack—Progress report on concrete corrosion studies

J. H. P. VAN AARDT, *Bulletin Series* DB 13, National Building Research Institute (Pretoria), Mar. 1955, pp. 44-60
Reviewed by D. G. NORMAN

Discusses laboratory experiments aimed at assessing resistance of portland cement mortars to aggressive agents.

Deterioration of concrete and reinforced concrete due to chemical and natural agencies

N. STUTTERHEIM, *Bulletin* No. 12, National Building Research Institute (Pretoria), June 1954, p. 5
Reviewed by D. G. NORMAN

Summarizes deterioration process, South African experience and research. Gives new theory for sulfate attack; mentions marine corrosion.

Excessive shrinkage of aggregates as a cause of deterioration of concrete structures in South Africa

N. STUTTERHEIM, *Transactions*, South African Institution of Civil Engineers (Johannesburg), V. 4, No. 12, Dec. 1954, pp. 351-367
Reviewed by D. G. NORMAN

Discusses aggregate shrinkage which caused sagging of beams over a fairly wide area in South Africa. Describes laboratory and field investigations.

Notes on the vulnerability of portland cement products to pure or soft water

Transactions, South African Institution of Civil Engineers (Johannesburg), V. 7, No. 9, Sept. 1957, p. 266
Reviewed by D. G. NORMAN

Describes work on National Building Research Institute. Soft waters were found to leach away cement.

Symposium on some approaches to durability in structures

Special Technical Publication No. 236, ASTM, 1958, 7 pp., $2.50

The symposium deals with nonmetallic materials that are generally used in structures where weather plays a major part in the durability of the structure. The problems of variations of kinds of exposure and of associated moisture migrations are considered. Papers cover testing and durability of concrete, durability tests of structural sandwiches, durability of buildings, effects of atmosphere on masonry, and relation between actual and artificial weathering.

Laboratory testing and the durability of concrete

THOMAS B. KENNEDY, pp. 14-26

Concrete structures cannot be constructed and subjected to full-scale tests for durability in a manner analogous to load tests. It therefore is necessary to estimate the durability of concrete structures by combining information gained from a study of service yielded by existing structures, controlled exposure tests of specimens, laboratory simulated-service tests, and tests and analyses of concrete and concrete materials. None of these sources of information provide a basis for adequately evaluating the effect on durability of workmanship, honesty, and intelligence that may or may not be employed in all stages of the operation from production of concrete materials to maintenance of the completed structure.

Petrographic examination of aggregates, x-ray diffraction studies of cement, freezing-and-thawing tests of concrete both in the laboratory and at field exposure stations, ultrasonic pulse velocity tests of specimens and structures, are recommended means of improving knowledge of durability of concrete and concrete structures.

Effect of the atmosphere on masonry and related materials

J. W. MCBURNEY, pp. 45-56 (including discussion)

Except for corrosion of metals, rather little attention has been given to air as a possible factor in disintegration of structural materials. The common assumption has been that all the deleterious substances which in time produce disintegration or excessive volume change in masonry were present originally in the units

or in the mortar ingredients as delivered and used. This paper considers the effect on masonry of such components of the air as carbon dioxide, alkali and alkaline earth salts, usually sodium, potassium, calcium and magnesium compounds, sulfur dioxide, and moisture as vapor expressed as relative humidity.

Cracking of concrete

F. A. Blakey, and F. D. Beresford. *Constructional Review* (Sydney), V. 32, No. 2, Feb. 1959, pp. 24-28

Reviews the classification of cracks in concrete. These are brought about by three general processes: "prehardening," restrained shrinkage, and cracking under load.

Prehardening cracks are known as "presetting," "plastic," or "settlement" cracks. These cracks are formed when settlement of the aggregate and cement pastes meet an obstruction such as reinforcement or a bridge of coarse aggregate. The time of cracking is usually about 20 min after placing.

Restrained shrinkage is brought about after bleeding and while the concrete is gaining strength. If shrinkage is restrained then tensile stresses will develop and cracks may appear.

Cracking under load begins in one of the following ways: in the paste, in the aggregate, or between paste and stone.

Statistical quality control of concrete and concrete products

K. W. Day. *Commonwealth Engineer* (Melbourne), V. 46; No. 5, Dec. 1958, pp. 46-48; No. 6, Jan. 1959, pp. 54-57; No. 7, Feb. 1959, pp. 73-75

A basis for testing and a specification which provides a tolerance for the manufacture between unusable and unsatisfactory products.

The second part in this series deals with the application of statistical control. The use of a large size graph with "compressive strength" versus "date of cast" provides at a glance the result of any days work in that month with regard to the upper and lower limits of strength. Further tests in workability, water content, specific surface of sand, curing temperature, and the Schmidt hammer test help not only in graphing these variables but results in closer control of the concrete which is further increased with statistical analysis of these results.

Part 3 outlines the need and method of obtaining reliable test data for use in statistical quality control. The kind of tests made were: concrete strength, water-cement ratio, workability, and aggregate grading. The applicability and limitations of these tests are discussed.

Vibration waves in concrete

N. S. Farrar. *The Engineer* (London), V. 206, No. 5354, Sept. 5, 1958, pp. 378-380; (discussion: Oct. 24, 1958, p. 646, and Dec. 5, 1958, p. 888)
Reviewed by Aron L. Mirsky

Reports continuation of Plowman's experimental work (*The Engineer*, Feb. 26, 1954, pp. 302-303, and May 31, 1957, pp. 830-832; "Current Reviews," ACI Journal, V. 26, No. 2, Oct. 1954 (*Proceedings* V. 51), p. 216, and V. 29, No. 5, Nov. 1957 (*Proceedings* V. 54), p. 440 and outlines development of a theoretical approach to the manner in which the particles of the mass of concrete undergo excitation. Author's theory of the decay of vibration waves is based on the impedance method of analysis. Correspondence between experimental and theoretical curves for decrease of acceleration with height above base became quite good after an initial period of random particle vibration. Author draws only one specific conclusion, that the vibration waves are nonlinear, pending more comprehensive experiments.

Accelerated curing tests on concrete

N. N. B. Ordman and N. G. Bondre. *Engineering* (London), V. 185, No. 4798, Feb. 21, 1958, pp. 243-245
Reviewed by Aron L. Mirsky

After brief description of oven-heating method of accelerated testing of concrete, developed under J. W. H. King, authors describe site laboratory set up by the Port of London Authority at Tilbury Docks with the two main purposes of performing normal concrete control tests in conjunction with a contract for construction of a terminal, and of using and developing the new technique. Results of development work on heating cycles and the maturity-strength law are reported. Authors note, among other things, that savings in cement due to mix modifications made possible at the Tilbury Docks site by closer control of concrete quality, using this technique, has far exceeded the cost of the laboratory.

The problem of how best to heat the specimens ("steamers" versus other methods)

soon brought the discussion to a boil. It appears in the issues of Mar. 14, 1958, p. 324; Mar. 21, p. 355; Apr. 4, p. 419; Apr. 11, pp. 451-452; May 23, pp. 666-667; June 20, p. 771; July 4, p. 3; and July 11, 1958, p. 36.

Corrosion of reinforcing steel in concrete in marine atmospheres

D. A. Lewis and W. J. Copenhagen, *South African Industrial Chemist*, V. 2, No. 10, Oct. 1957
Reviewed by D. G. Norman

Corrosion on Natal Coast in South Africa is severe. Authors are currently engaged in investigating this problem.

Concerning the nature of the so-called protection against shear (Über das Wesen der sogenannten Schubsicherung)

G. Kani, *Der Bauingenieur* (Berlin), V. 33, No. 10, Oct. 1958, pp. 375-382
Reviewed by Aron L. Mirsky

Author defines *Schubsicherung* as measures taken to provide security against damage due to shear (following Mörsch, this means increasing the ultimate strength of the beam without increasing the dimensions of the cross section or the amount of main reinforcement). He points out that shear occurs on oblique planes in concrete columns axially loaded in pure compression, yet nobody worries about it. He also describes Seybold's (1933) experiments on shear, both proper and diagonal tension, in brittle materials, which led to the same conclusion as stated by other investigators: the concrete ruptures in tension before its ultimate strength in shear is reached.

Major portion of paper is devoted to problem of shear in prestressed concrete beams with and without bond, and in the prestressed beam with bond but without shear reinforcement (the simply reinforced beam is thus considered a special case of this last, with the prestressing force reduced to zero). Author concludes that the so-called *Schubsicherung* in reality has nothing to do with protection against shear stress, but concerns protection via bonding, and that simply reinforced beams, even those wherein the reinforcement is provided with deliberate anchorage in the form of hooks, have lower ultimate strengths than prestressed beams with bond, because of the additional anchorage of the steel in the latter.

Special refractory concretes (in Hungarian)

A. Cser, *Epitoanyag*, V. 9, No. 6, 1957, pp. 293-297
Hungarian Technical Abstracts V. 10, No. 3, 1958

Reports on research conducted on refractory concretes which have a ceramic bond rather than hydraulically bonded aggregates. The bonding of corundum concrete is treated in detail and the binding process and physical characteristics of concretes with oxysulfate and phosphate bonds are also treated.

Structural Research

Bond of reinforcement to concrete (Adhérence des armatures au béton)

R. Dutron, *Revue des Matériaux* (Paris), No. 514-515, July-Aug. 1958, No. 516, Sept. 1958, pp. 223-230; No. 517, Oct. 1958, pp. 257-264
Reviewed by Phillip Melville

Bond strength either normal or tangential was measured in modified pull-out tests. Bond was found to be more closely related to flexural than the compressive strength of concrete and mortar. Bond is a physical phenomenon helped by concrete and mortar shrinkage (tangential bond is also increased by roughness) and was found, for a given mortar and concrete strength to be higher for vertical than horizontal bars. For plain bars the bar diameter was found to be without effect on bond strength and the coefficient of friction between steel and concrete was found to be $\mu = 0.5$.

Investigations on a model dome with arched cut-outs

L. K. Stevens, *Magazine of Concrete Research* (London), V. 11, No. 31, Mar. 1959, pp. 3-14
Author's Summary

Experimental investigations on a model dome of varying thickness and with large arched cut-outs around the base are described.

These investigations include the determination of strain distributions under the action of uniform and nonuniform conditions of moment and thrust at the base, and the confirmation of an approximate method of analysis developed to predict the structural behavior.

Shear tests on joints between precast post-tensioned units

L. L. Jones, *Magazine of Concrete Research* (London), V. 11, No. 31, Mar. 1959, pp. 25-30
Author's Summary

Shear tests on three types of joints between precast post-tensioned units has resulted in data from which the minimum shear strength of these joints may be calculated. The minimum value of the coefficient of friction of the surfaces at a plain butt joint was found to be 0.391 and that of the surfaces at a mortar joint 0.645.

The results obtained when testing rejoined cracked members are discussed, and an analysis is made which is consistent with the mode of failure.

Comparison of the behavior of concrete beams under static and dynamic loading

J. J. Trott and E. N. Fox, *Magazine of Concrete Research* (London), V. 11, No. 31, Mar. 1959, pp. 15-24
Author's Summary

Describes preliminary experiments to determine whether or not the rate of loading significantly affects the mechanical properties of concrete beams tested in flexure. Two rates of loading were used: 5×10^{-7} in. per in. per sec, the rate employed in normal mechanical tests; about 6×10^{-2} in. per in. per sec when the beams were tested under impact conditions.

If the onset of cracking is associated with the departure from linearity of the stress-strain curve, it appears that increase in the rate of loading markedly effects the strain at which cracking begins.

Model experiments on diagonally interconnected flat roofs in domes

Peter Smith, *Architectural Science Review* (Sydney), V. 2, No. 1, Mar. 1959, pp. 33-38

Article discusses tests and the results for four different plans for spanning a square area. The material used for the model was perspex (methyl methacrylate) plastic. The object was to compare diagonally interconnected beams with rectangularly interconnected beams to span the area. Four different types of frames were used: (1) diagonally interconnected beams, eight bays with edge beams; (2) diagonally interconnected beams, eight bays without edge beams; (3) diagonally interconnected beams, six bays without edge beams; (4) rectangularly interconnected beams, six bays. All based on spanning an area of 40 ft x 40 ft supported at corners. Also conducted test on a model of a dome over 45 ft in diameter. Concludes that the diagonally interconnected frame with edge beams and the rectangularly interconnected frame, having approximately the same bay size, require the same amount of material.

Experiments made with wall elements of large dimensions in a large plate building prototype (in Polish)

Z. Cwiok, S. Zaleski, and S. Zielinski, *Inzynieria i Budownictwo* (Warsaw), No. 2, 1958, pp. 47-51
Polish Techincal Abstracts
No. 3 (31), 1958

Reports on experiments that were conducted to determine ultimate bearing strength and deformations of eccentrically and axially loaded walls. Data obtained from these experiments have led authors to make certain suggestions relating to bearing strength, amount and distribution of reinforcement, and behavior of such elements under loading.

How to make conductive terrazzo flooring

P. J. Sereda, *Canadian Builder* (Ottawa), V. 8, No. 8, Aug. 1958, pp. 39-40; reprinted as NRC 4903; available from the National Research Council of Canada, Ottawa

Deals with the design and construction of a conductive terrazzo flooring for use in operating rooms in a hospital. Results are given of the performance of this type of flooring. This article is intended to serve as the basis for a specification for this type of flooring.

Tests with cantilever retaining walls

A. Kurylo and J. Hradiuk, *Bauplanung und Bautechnik*, V. 11, No. 12, 1957, pp. 526-532. Reviewed in *Building Science Abstracts*, V. 31, No. 6, June 1958, p. 175
Highway Research Abstracts
Mar. 1959

From the tests carried out formulas are presented for determining the geometrical dimensions of cantilever retaining walls on a rigid bed. Three phases can be detected in the performance of the vertical reinforced concrete wall: the first from the application of loading to 80-100 percent filling; the second from 80-100 percent filling to the appearance of the first cracks in the concrete; the third from the initial cracking to rupture.

Loss of stability in walls is due to tilting (walls on an elastic bed). Wedges of soil form both on tilting and slip of cantilever retaining walls. On account of the presence of the wedge at rupture, the usual method for the practical control of the stability of cantilever retaining walls can be regarded as fully satisfactory.

Safety against anchorage failure in structures without shear reinforcement (in Swedish)

ARNE HILLERBORG, *Nordisk Betong* (Stockholm), V. 3, No. 2, 1959, pp. 119-138

Reviewed by MARGARET CORBIN

Discusses a theoretical and experimental study of the risk of anchorage failure in reinforced concrete structural units without shear reinforcement. Study reveals that the classical assumptions used in the calculation of bond stresses do not yield an accurate picture of the mechanism of failure or of the ultimate load. A tentative design method is proposed based on a schematic representation of the mechanism of failure which takes into account the definite slope of cracks and the presence of uncracked zones. It is proposed to substitute this method for the procedure based on checking bond stresses used today. Tests include simply supported beams under concentrated loads and cantilever beams under a uniformly distributed load simulating footings. Conclusions: (1) classical theory diverged widely from test results; (2) standard (Swedish) design yielded factors of safety of about 1.5 to 10 whereas the appropriate value seems to be 3; (3) values obtained by the proposed tentative method conformed more closely to test results obtained.

General

The reinforced concrete dome of the Melbourne Public Library, 1911

DAVID A. L. SAUNDERS, *Architectural Science Review* (Sydney), V. 2, No. 1, Mar. 1959, pp. 39-46

Melbourne once had the largest reinforced concrete dome in the world, but most designers are not familiar with it. Author intends to publicize this dome somewhat and gives history of the conception, design, and construction of the dome. The dome itself was placed during the months of May and June 1911. It is still standing and shows no defect in the main structural members. Copies of drawings show architectural plans and sections and structural details.

Architecture to reflect a process—Flow line food production in a factory built for Heinz

Engineering (London), V. 187, No. 4860, May 1, 1959, pp. 576-578

Reviewed by ARON L. MIRSKY

Let it be admitted right off that concrete does not play the dominant role in this bean and soup factory, but not for reasons reflecting adversely on its characteristics.

Point made by Michael Ryan, an architect, is an interesting one (as to its merits, reviewer, who classes himself with the engineers, reserves judgment*): the engineer tends to be a specialist; any major structure requires both a coordinator and a designer trained to look for and design beauty *into* a structure (rather than adding gingerbread as an afterthought); the obvious person to serve in this managerial capacity is the architect. As relevant and material evidence, Mr. Ryan cites the new Heinz factory at Kitt Green, Wigan, where site peculiarities, production flow requirements, employee amenities, and other factors were coordinated by the architect, with the cooperation of the various engineers, into a harmonious, efficient, aesthetically pleasing whole.†

Durability and maintenance of concrete structures on Danish railways

ARNE JEPPESEN, *Progress Report* B3, Committee on Alkali Reactions in Concrete, Danish National Institute of Building Research and Academy of Technical Sciences, Copenhagen, 1958, 75 pp., 12 D kr

The author has compiled information and data on the durability of concrete structures of the Danish State Railways, the techniques of repair as they have been developed during the last couple of decades, and an analysis of maintenance and repair costs versus initial costs for the period 1900-1950.

*He merely calls attention to the trenchant editorial in the same issue (p. 565), titled "Who Cracks the Whip?", and to an even more robustly-worded editorial in *Concrete and Constructional Engineering* for May, 1959.

†A much more prosaic account of this facility, *sans* controversial elements (for those who prefer it that way), will be found in *The Engineer* (London), V. 207, No. 5391, May 22, 1959, pp. 809-812.

ACI News Letter

CONTENTS

October 1959

ACI 1960 Convention—New York City..... 1

Positions and Projects..... 5

Honor Roll.....13

New Members.....16

Tools, Materials, Service...18

Looking Ahead.....23

List of Advertisers.....32

56th Annual Convention New York City, March 14-17

An outstanding program for the 56th annual meeting of the Institute has been announced by the Technical Activities Committee and the general chairman of the New York local committee, Roger H. Corbetta, president of the Corbetta Construction Co.

Convention-goers, in session at the Commodore Hotel in the heart of Manhattan, March 14-17, will hear a technical program geared to be of interest to the designer, builder, research man, and those in fields related to the concrete industry.

The Institute's Board of Direction and Technical Activities Committee will begin their work ahead of the general sessions with meetings on Saturday and Sunday, March 12 and 13.

Monday, March 14, and Tuesday morning, March 16, have been designated "committee days" and numerous ACI technical committees will convene in open session to review current work and aims.

The first general session will be called to order on Tuesday, March 15, at 2 p.m. The first session will feature proposed ACI Standards and committee reports.

Concurrent sessions on March 16 will be devoted to model tests and highways in the morning, materials, and design and structural research in the afternoon. On March 17 Committee 201, Durability of Concrete in Service, Hubert Woods, Portland Cement Association, chairman, will present a symposium on restoration of deteriorated concrete. A concurrent session on design and construction will also be held Thursday morning.

Committee 115 will conduct its annual research forum on Thursday afternoon, March 17, present-

ing latest reports of tests on concrete in progress at institutions throughout the world.

Annual awards will be presented by President Phil M. Ferguson at the Awards Luncheon, Wednesday, March 16.

The local planning committee are arranging a number of "extras" to make the 56th annual meeting thoroughly enjoyable as well as worthwhile.

TAC holds summer meeting at Waterways Experiment Station

The ACI Technical Activities Committee met August 20-21 at the Waterways Experiment Station, Jackson, Miss.

Following the regular business session on Thursday, TAC was guest of the Waterways Experiment Station Friday at a luncheon at the Station cafeteria and a conducted tour of the facilities in Jackson and Vicksburg.

Col. Edmund H. Lang, director, greeted the group and described the functions of the Waterways Experiment Station. The inspection trip included a tour of the soils test section; a walk through work areas of Flexible Pavement Branch and Army Mobility Research Center; and a tour of several hydraulic models including Niagara River and Falls; St. Lawrence Seaway Project; Narrangansett Bay, and a number of scale model dam sites.

REVIEWERS NEEDED

Betongen Idag

This publication of Norsk Cementforening, Oslo (in Norwegian) is now being received regularly at Institute headquarters. A volunteer reviewer is needed to keep members informed of pertinent papers in this publication through the "Current Reviews" section of the JOURNAL. Those interested please write the Secretary of the Technical Activities Committee at ACI headquarters.

Memoires, Societe des Ingenieurs Civils de France

This publication, in French, is now being received regularly at Institute headquarters. A volunteer reviewer is needed to keep members informed of pertinent papers in this publication through the "Current Reviews" section of the JOURNAL. Those interested please write the Secretary of the Technical Activities Committee at ACI headquarters.

Beton Herstellung Verwendung

This German language technical periodical is now being received regularly at Institute headquarters. A volunteer reviewer is needed to keep members informed of pertinent papers in this publication through the "Current Reviews" section of the JOURNAL. Those interested please write the Secretary of the Technical Activities Committee at ACI headquarters.

Institute headquarters attracts architects

International interest in the American Concrete Institute was prominent this month with the "Visitors' Register" showing 30 architects from Paris, on tour with the French Association of Architects, visiting ACI headquarters recently.

ACI members Ernesto R. Montiel, Universidad del Zulia, and Alvaro Hernandez, Martin Engineering Co., both of Maracaibo, Venezuela, were among the many inspecting the building during the past month.

Walter retires from Bureau of Reclamation post

Donald S. Walter recently ended a professional engineering career with the Bureau of Reclamation which started nearly 40 years ago.

For the past 10 years Mr. Walter has been regional engineer for Region 1 of the Bureau of Reclamation, which encompasses the states of Idaho, Washington, Oregon, western Montana, and western Wyoming. In this position, Mr. Walter exercised primary staff responsibility over the design and construction functions of such major reclamation projects as the Columbia Basin and Yakima projects in Washington, the Hungry Horse Dam in Montana, the Deschutes and Crooked River projects in Oregon, and the Palisades, Boise, and Minidoka projects in Idaho.

A member of ACI since 1941, Mr. Walter has been active in Institute technical committees for many years. He recently served a 3-year term as chairman of ACI Committee 210, Resistance to Erosion in Hydraulic Structures, discharged February, 1959.

PRESTRESSED CONCRETE BEAMS for U.S. Route 1 Bypass . . .

Cranes lower prestressed concrete beam 95' long, 4' square, and weighing 58 tons, onto piers of one of the middle spans of the 4-span Ridley Creek Bridge. There are 16 beams per span. Note "railway" across creek at bottom of picture, used to trolley one end of beam across creek to crane at right.

Owner:
Penna. Dept. of Highways

Engineers:
Harris-Dechant Associates
and McCormick-Taylor Associates,
both of Philadelphia, Pa.

General Contractor:
Glasgow, Inc.
Glenside, Pa.

Contractor for Construction of Bridges:
McClain Construction Co., Inc.
Philadelphia, Pa.

Manufacturer of Prestressed Beams:
New Castle, Del. Plant,
Atlantic Prestressed Concrete Co.
Trenton, N. J. (wholly-owned
Subsidiary of Warner Company,
Philadelphia, Pa.)

Ready Mixed Concrete for Beams Supplied By: Warner Company
Philadelphia, Pa.

Placing one of shorter beams for end span at Ridley Creek. Mobile crane (out of picture to right) is moving beam on rail dolly out to meet crane stationed on creek bed below. The two cranes then move beam to final position.

• Ridley Creek Bridge, pictured here, is one of seven bridges required in the 8 mile long bypass around Media, Pa. Over 400 prestressed box beams, ranging in length from 40' to 95', were used in these structures.

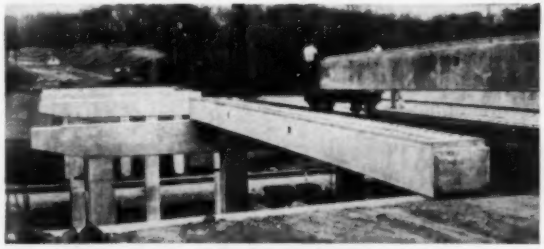
Factors influencing the choice of this modern bridge construction technique were low initial and maintenance costs, ease and speed of erection, and the trim appearance of prestressed concrete.

In manufacturing the prestressed beams for these bridges, Atlantic Prestressed Concrete Company used Lehigh Early Strength Cement for maximum production efficiency. Units were completed quickly, ready for trucking to the job site as needed.

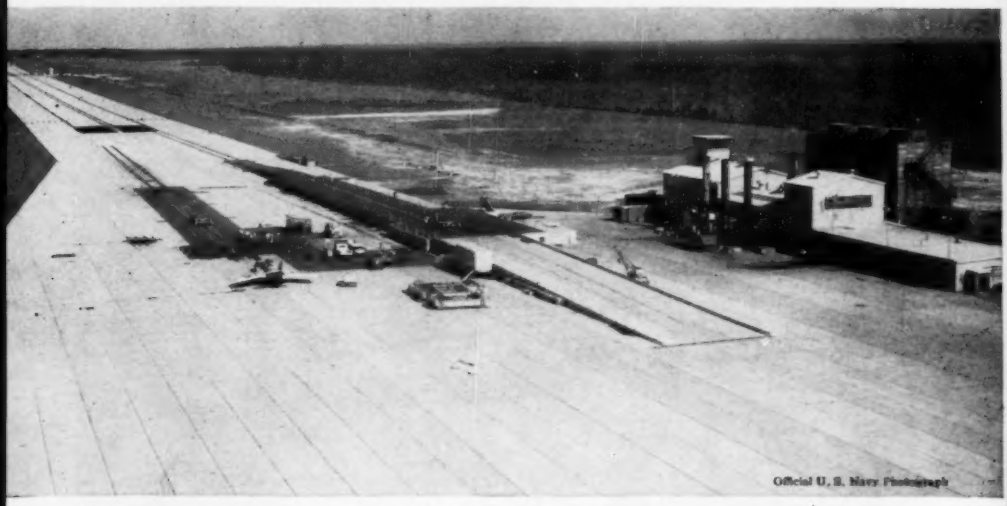
This is another example of the advantages of Lehigh Early Strength Cement in modern concrete construction.

LEHIGH PORTLAND CEMENT COMPANY

ALLENTOWN, PA.



POZZOLITH . . . makes good concrete better



Official U. S. Navy Photograph

NAVAL AIR TEST FACILITY—Lakehurst Naval Air Station, N. J. Construction Agency: Bureau of Yards and Docks of the Navy Department. Constructed under the direction of the District Public Works Officer, 4th Naval District, Philadelphia, Pa.

• Consulting Engineer on Pavements: Louis Berger & Associates, Orange, New Jersey • Contractor: Roscoe Engineering Corporation & Associates, Washington, D. C. • Pavement Sub-Contractor: S. J. Groves & Sons Company, Woodbridge, N. J.

900 psi flexural strength— POZZOLITH helps meet this requirement at The Naval Air Test Facility

The Runway Arrested Landing Site of the Navy's recently completed $23,000,000 test facility required concrete of unusually high quality.

To withstand anticipated stress of jet planes simulating aircraft carrier landings, reinforced concrete having a flexural strength of 900 psi was specified.

Preliminary mix designs—with cement factors ranging up to 8½ sacks per cubic yard and with the water-cement ratio of 4.0 gallons per sack—did not provide a workable mix with the required high flexural strength.

The following mix was then employed:

Cement, Type 1—729 lbs.—(7¾ sks.)
Sand (SSD)—1,000 lbs.
C.A.—Trap Rock—
(Houdaille, Ind., Bound Brook, N.J.)
¾"—895 lbs. 1½"—1342 lbs.
Water (total)—28 gal. (3.62 gal./sk.)
POZZOLITH

This mix was workable and developed a flexural strength of 951 psi in 7 days and 1114 psi in 28 days. Compressive strength averaged 8537 in 28 days.

Because of these high strengths the water-cement ratio was later increased to 3.75 and the cement factor reduced to 7½ bags. Flexural strength of the revised POZZOLITH mix averaged 981 psi in 28 days.

For further details on the performance of POZZOLITH on this and other airfield projects write to—

The Master Builders Company, Cleveland 3, Ohio
Division of American-Marietta Company
The Master Builders Co., Ltd., Toronto 15, Ont.
International Department, New York 17, N.Y.
Branch Offices in all principal cities.

MASTER BUILDERS POZZOLITH*

**POZZOLITH* is a registered trademark of The Master Builders Company for its concrete admixture to reduce water and control entrainment of air and rate of hardening.*

Positions and Projects

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 336, Combined Footings

W. Fuchssteiner
Consulting Engineer
Darmstadt, Germany

Elwyn E. Seelye
Seelye, Stevenson, Value and Knecht
New York, N. Y.

Committee 337, Strength Evaluation of Existing Structures

Charles W. Yoder
Charles W. Yoder & Associates
Milwaukee, Wis.

Committee 712, Precast Structural Concrete Design and Construction—Joint ACI-ASCE

Kenneth C. Naslund
The Engineers Collaborative
Chicago, Ill.

Committee 717, Practice in Low-Pressure Steam Curing

Edward Mangotich
Arizona Precast Concrete Co.
Phoenix, Ariz.

R. J. Sweitzer
Lock Joint Pipe Co.
Wharton, N. J.

American-Marietta acquires Cedar Rapids company

Concrete Materials and Construction Co., Cedar Rapids, Iowa, supplier of crushed stone, sand, and gravel, has been acquired by American-Marietta Co.

Grover M. Hermann, chairman of American-Marietta, reported that the Cedar Rapids company and its associated firm, Concrete Materials Company, will be operated as an independent Division of American-Marietta. There will be no changes in its management, personnel, or facilities.

McGinnis appointed chairman of ACI Committee 402

Howard J. McGinnis, Structural and Railways Bureau, Portland Cement Association, Chicago, has accepted chairmanship of Committee 402, Concrete Floor Finishes. Mr. McGinnis is a member of ACI Committee 401, Specifications for Structural Concrete.

IBPC selects Stockholm as site for 1960 congress

The international organization of the precast concrete industry, International Bureau for Precast Concrete, will hold its third international congress in Stockholm, June 16-22, 1960 at Sankt Erikshallen.

The association of the Swedish precast concrete industry, Cementvarufabrikernas Riksforbund, has arranged the meeting. Agne J. Sandberg, president of the international organization, is serving as chairman of the congress committee.

The preliminary program includes 3 days of working sessions and discussions; 2 days for visits to industries, building projects and modern residential areas in and near Stockholm; excursion by boat in the archipelago of Stockholm; and other social events.

Immediately adjoining the congress hall there will be an international exhibition including finished concrete products of various types and the raw materials embodied, as well as machinery and equipment used for production.

Details available from the Congress Secretariat, P.O. Box 14045, Stockholm 14, Sweden.

Rockwin forms prestressed concrete consulting group

Rockwin Prestressed Concrete Corp., Santa Fe Springs, Calif., announces the formation of an engineering consulting group, Rockwin Engineers, exclusively for specialized service in the prestressed concrete field. Services will be available to architects and engineers only.

Steven Galezewski, a pioneer in prestressed concrete in this country, heads up the group.

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National paving record set in Michigan

Michigan recently laid claim to a national record for laying pavement on the Detroit-Chicago expressway. The Highway Department announced that the Pierson Construction Co., Saginaw, placed 6242 ft of 24-ft wide, 9-in. thick concrete on the Detroit-Chicago expressway, US-12, in a 12-hr. working day near Watervliet in Berrien county.

The previous 1-day record was set by the same contractor on US-12 on July 17 when 6050 ft were placed.

Over 100 miles of the 220-mile Detroit-Chicago expressway are currently under construction and another 80 miles are open to traffic.

Lee leaves Concrete staff

Douglas Lee, editor since October, 1956, has left *Concrete* magazine. Mr. Lee is now associated with an advertising agency, Perrin-Paus Co., Chicago.

New editor of the magazine is Jack Anderson, former associate editor of *Brick and Clay Record*.

Guntin joins Sika technical sales staff

Daniel Guntin has joined the technical sales staff of Sika Chemical Corp., Passaic, N. J. He will serve architects, engineers, and contractors in the metropolitan New York area.

Active in the concrete industry, Mr. Guntin is a member of the American Concrete Institute, The Concrete Industry Board of New York, and the N. Y. Chapter of the Concrete Specifications Institute.

Sendker named New England manager for Alpha Cement

Francis T. Sendker has been named New England district sales manager by Alpha Portland Cement Co., Easton, Pa. His appointment follows the September 1 retirement of Melville B. Chard who formerly held the position.

Mr. Sendker, who will occupy Alpha's New England headquarters in Boston, was formerly sales representative for the company in the Syracuse area. He holds a civil engineering degree from Syracuse University and is a member of ACI and ASCE.

Carter succeeds Spears as PCA Salt Lake City district engineer

The Portland Cement Association has announced the appointment of Alan C. Carter as Salt Lake City district engineer. He succeeds Ralph E. Spears, who has been named Regional Structural Engineer for the association's west central region.

Huron Cement to erect new engineering building and lab

Plans to build a new 24,000 sq ft building at the Alpena, Mich., mill of the Huron Portland Cement Co. were announced recently. The new building will house the engineering, laboratory, and personnel departments.

The 2-story structure will be erected inside the main gate. Construction is expected to start early this fall with completion scheduled for the spring of 1960. The building was designed by Harry Fleischer of New York City.

ACI Records System Revised

Effective about Oct. 1, 1959, ACI membership records will be converted to the IBM punched card system of record maintenance. This decision by the Executive Committee was prompted to assure speedier and more efficient handling of membership records at less expense.

Under the new system, dues invoices, and JOURNAL and other large mailings will be handled mechanically in a fraction of the time now required.

Should your JOURNAL fail to reach you, or is misaddressed, due to the records changeover, please notify ACI headquarters so prompt correction can be made.

Darex Diary

by Mel Prior
Member, American Concrete Institute
Technical Service Manager, Construction Products
W. R. GRACE & CO.
Dewey and Almy Chemical Division
(No. 17 of a series)

"SLOW MOTION" FOR BETTER CONCRETING

RETARDING the set of concrete has many other purposes besides overcoming the difficulties caused by hot weather. Retardation is particularly useful when large volumes of concrete are to be placed, or when the placing must be done over a longer period than usual. Proper retardation helps prevent such bad results as cold joints, thermal volume change resulting from too rapid a rise in temperature, and the failure of the concrete to bond to reinforcing steel.

On big jobs, there are often long waits between placements because of widely-spaced deliveries, time required to move forms, and problems with difficult placements. In situations of this kind, the surface of the first-placed load of concrete may set before the next load is ready. When this happens, a "cold joint" results—there is little or no bond between the two placements of concrete. This weakens the structure and provides a channel for the flow of water, causing further damage. This problem can be overcome by using DARATARD to slow the setting rate of the concrete, which then remains plastic until the second placement is made. This results in a firm bond between the two.

When concrete sets rapidly, its temperature often increases greatly, which can cause cracks, with weakened concrete as the end result. This process is intensified in large volumes of concrete. Three factors are involved: the amount of heat generated by the hydration process, the rate at which this heat is generated, and the speed at which it is dissipated. Normally, the rate of heat generation is far greater than the rate of dissipation, so that heat builds up in the concrete, causing cracks. With the proper use of DARATARD, the rate of heat generation can be reduced almost as low as the dissipation rate, so that there is no great amount of heat built up in the concrete at any one time, and cracks are prevented. Not only is cracked concrete inherently weak, but water and water-bearing salts can penetrate the cracks and weaken the concrete even further. The proper use of DARATARD prevents the original cause of this cycle of difficulties and greatly reduces the risk of such concrete distress.

In bonding concrete to steel reinforcements, there are many situations in which the bond will break because the concrete sets too fast. Bridge slab construction is a typical example. As concrete is placed on the bridge girders, they bend under the weight. The concrete slab itself also moves under its own increasing weight as more and more concrete is placed. If the first-placed concrete sets and therefore loses its plasticity before the last of the concrete is placed, the total weight of the complete placing can cause movement beyond the elastic limit of the first-placed part of the total. The newly formed bond to the reinforcing steel will be broken and cracks will develop in the slab. The concrete will thus be weakened on two counts.

Here again, DARATARD can prevent such damage by preserving the plastic properties of the first-placed concrete until the entire slab has been placed.

The water reducing properties of retarders can also be used to good advantage in maintaining adequate workability with a minimum water content. This feature is particularly advantageous in hot weather. It is well known that concrete strengths suffer in hot weather not only because of increased water demand, but also due to the inherent effect of temperature on proper strength development. Hence, any steps that can be taken to improve strength under such conditions should not be overlooked. DARATARD will accomplish this task.

W.R. GRACE & CO.
DEWEY AND ALMY CHEMICAL DIVISION

Cambridge 40, Mass.; San Leandro, Calif.; Chicago 38, Ill.; Montreal 32, Canada

Day retires after 54 years in architectural field

Clarence E. Day, vice-president and director of Harley, Ellington and Day, Inc., Detroit architects and engineers, has retired after 54 years of service in the architectural field.

He became a partner of Harley, Ellington and Day in 1939 and was made vice-president in 1942. He will continue to serve the firm on a consulting and advisory basis.

Three ACI members form new consulting service

Three prominent structural engineers, Donald E. Anderson, Jack R. Janney, and Kenneth C. Naslund, have selected Chicago as headquarters for their new consulting service, The Engineers Collaborative, which offers nationwide coverage. The new firm also maintains a model analysis laboratory in Des Plaines, Ill.

Mr. Anderson and Mr. Naslund were formerly associated with Skidmore, Owings and Merrill as assistant chief structural engineer and chief structural engineer, respectively. Mr. Naslund is a member of ACI-ASCE Committee 712, Precast Structural Concrete Design and Construction.

Mr. Janey has been an independent consultant since 1956. Prior to that time he was associated for many years in the Research and Development Laboratories of the Portland Cement Association. He is chairman of ACI-ASCE Committee 712, Precast Structural Concrete Design and Construction, and a member of ACI-ASCE Committee 323, Prestressed Reinforced Concrete.

Stone and Webster appoint three assistant vice-presidents

Appointment of three assistant vice-presidents of Stone and Webster Engineering Corp., two in Boston and one in New York, has been announced by T. Cortlandt Williams, president of the world-wide engineering and construction organization.

The new assistant vice-presidents are Robert S. Boyd, appraisal manager; J. Marshall Hamill, administrative assistant in Boston, and Wilbur S. Roberts, Jr., manager of new business activities of New York.

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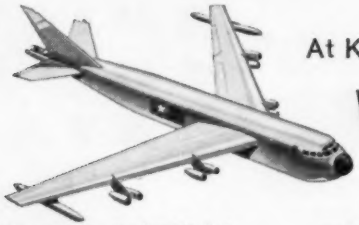

At Kinross Air Force Base . . .

World's heaviest

THE "JET AGE" IS HERE. And with it is the problem of re-building airports to provide the longer, stronger, heavier runways these big jets need. That's why airport builders and designers are planning thicker, stronger airfield pavements with extra-heavy reinforcement . . . pavements that will match or exceed present jet requirements. And that's why the heaviest welded wire fabric ever produced was installed at the vital Kinross Air Force Base in Michigan.

Operational jets in commercial service now impose gear loadings in the neighborhood of 130,000 lbs., with gross weights of 295,000 lbs. Future models may go even higher. For that reason, only reinforced concrete provides the added strength to meet the unusually heavy and severe requirements of the "jet age." Specify USS American Welded Wire Fabric for reinforcing all runways, high speed turnoffs, taxiways, and service pads . . . get these important advantages:

- USS American Welded Wire Fabric distributes heavy wheel loadings over a larger area to prevent harmful overloads at any one point. Impact, shock, suddenly applied or released loads such as occur during landings and take-offs are more uniformly "absorbed" by each slab. Reinforced concrete slabs are 30% stronger than unreinforced slabs of equal thickness.
- USS American Welded Wire Fabric prevents harmful cracking from developing because of its uniformly spaced high-tensile-strength welded steel wire construction. Its action "knits" a slab together.
- USS American Welded Wire Fabric unifies the action of any slab when it expands and contracts due to temperature change.
- Longer, wider slabs can be laid when USS American Welded Wire Fabric is used. This means fewer joints, fewer load transfer assemblies. The reduction in the number of joints not only means economy, but also a smoother, safer landing surface.

When existing airfield pavements must be strengthened, USS American Welded Wire Fabric reinforced concrete overlays will produce unbroken grade lines and provide additional strength. Thus, maximum economy is achieved.

Specify USS American Welded Wire Fabric. It's available in a wide variety of styles, lengths, and widths . . . in wire sizes from ½" diameter to 16 ga. and in longitudinal or transverse wire intervals of 2" to 16". The engineers at American Steel & Wire will be glad to tell you more about the application possibilities of fabric and how it can serve your needs. Get in touch with American Steel & Wire, Dept. 9207, 614 Superior Avenue, N.W., Cleveland 13, Ohio.

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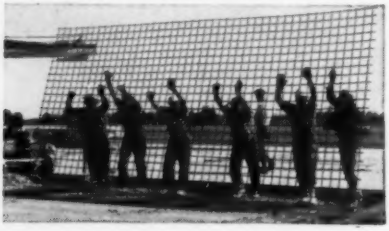

Contractor: Loselle Construction, Inc., Wyandotte, Michigan
Paving Material Supplier: Scioto Supply Company, Lansing, Michigan
Design and Supervision: Corps of Engineers, U. S. Army

At Kinross Air Force Base, workmen are installing a section of USS American Welded Wire Fabric—Style 55-7/07/0 (7/0 Gauge is 0.490" Diameter Wires). USS American Welded Wire Fabric is entirely fabricated by electrically welding all intersections of the high strength steel wires. This insures positive placement of the steel in the slab and eliminates costly field tying and extra handling.

Welded Wire Fabric helps the big jets land safely!

Under the direction of the U. S. Corps of Engineers, another new United States Air Force Base is made ready to join our air defense network. It is Kinross, located on the upper Peninsula of Michigan. This base was expanded to handle bigger, heavier modern jet aircraft. USS American Welded Wire Fabric was used in the rigid overlays on the strengthened and lengthened runways.

USS American Welded Wire Fabric

LOS ANGELES INTERNATIONAL AIRPORT,
COMPLEX I, EAST-WEST RUNWAYS
Los Angeles, California
Owner: City of Los Angeles
Architect: Pereira and Luckman, Paul R. Williams, Welton Beckett & Assoc.
Engineer: Donald B. Warren Co. and S. B. Barnes
Contractor: Griffith Company
Laboratory: Bureau of Standards, City of Los Angeles
Maracoconcrete: Griffith Company
Structure: Paving concrete

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B. Decrease permeability.
C. Achieve greater density and higher durability factors.

2 LOWER CONCRETE COSTS: —

A. Maintain slump and workability at low W/C ratios.*
B. Attain higher strengths without increasing cement content of a mix.**
C. Permit economical redesign of conventional concrete mixes.

*Decrease water by as much as 20% . . .
**Increase strengths by as much as 40% . . . depending on the mix design.
(The above figures are not represented as maximum. Greater percentages have been achieved in both respects.)

MARACON also reduces water requirements in concrete mixes containing pozzolanic materials.

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CHEMICAL SALES DEPARTMENT
MENASHA, WISCONSIN

Carolina Solite expands facilities at Aquadale

Construction of a fifth 135-ft rotary kiln at Carolina Solite's Aquadale plant has recently been completed. This has been the third expansion of production facilities at Aquadale since the plant began operations in 1953.

Sommerschield affiliates with Stanley Engineering

Harold F. Sommerschield, formerly a partner in the architectural engineering firm of Suter and Sommerschield, Chicago, has become associated with the Stanley Engineering Co., Chicago.

47th National Safety Congress, Chicago, October 19-23

"Safety in the Sixties"—a forecast of things to come in safety in the next 10 years—will be the theme of the 47th National Safety Congress, annual convention of the National Safety Council.

The Congress, the world's biggest safety show, will be held in Chicago, October 19-23 with headquarters at the Conrad Hilton Hotel.

Master Builders names Neuman Detroit branch manager

Jack E. Neuman has been named manager of the Detroit branch office for The Master Builders Co., according to an announcement by Stephen W. Benedict, president. He fills the position recently vacated by Charles Lyon who has been named special assistant to the vice-president for marketing.

Holforty establishes consulting office

Clifford W. Holforty, Rochester, Mich., has announced his entrance into private practice as a consulting engineer in the fields of structural and civil engineering, site planning, land development, traffic analysis, and parking lot design.

Honor Roll

January 1—September 30, 1959

Membership in ACI offers a professional exchange of ideas, methods, and procedures dealing with concrete and concrete construction. It is through this exchange that each individual receives an ever increasing technical knowledge. To broaden this scope of knowledge, get your friends and acquaintances to join the American Concrete Institute.

Robert P. Witt	52½
Samuel Hobbs	44
George B. Southworth	23
Alfonso Marin E.	18
Louis A. Gottheil	13
David A. Sauer	13
Faraj Tajirian	13
Robert F. Adams	12
James Chinn	12
Newlin D. Morgan, Jr.	12
Joe W. Kelly	11
Charles C. Luther	10
H. C. Pfannkuche	10
Chas. W. Cole, Jr.	9
Phil M. Ferguson	9
Kenneth M. Huber	9
A. T. Klassen	9
Francesco Sardella	9
Joseph J. Shideler	8½
Wendell H. Nedderman	7
Glenn C. Thomas	7
Henry Aaron	6
Jose Maria Bravo	6
John L. Burdick	6
S. D. Burks	6
Ward Dobbin	6
Alberto Dovali Jaime	6
Myron L. Goral	6
Mauro N. Guzman	6
John E. Heer, Jr.	6
Gregorio Hernandez	6
Horace G. Hill, III	6
Aleck E. Hiscox	6
Truman R. Jones, Jr.	6
Elias Kardaras	6
George J. Kerekes	6
George E. Large	6
Leo M. Legatski	6
Frank B. May	6
R. C. Postlethwaite	6
George N. Scofield	6
L. J. Sewell	6
Ferruh Taskin	6
John P. Thompson	6
George Batievsky	5
W. S. Cottingham	5
Victor M. Gallo	5
Jack C. McCoe	5
Abef Moreno Przespolewski	5
W. E. Moulton	5
Gene M. Nordby	5

Precast Concrete Components Lifted and Placed with Richmond Lifting Inserts

Precast wall slab being tilted into place by means of Richmond Lifting Inserts and Lifting Brackets.

Certified tests by an independent laboratory assure proper strength rating for efficient design and performance in concrete at usable strength

Richmond has developed and tested a complete line of Lifting Inserts and accessories for handling and placing of precast concrete wall slabs, columns, beams, girders, piles, etc.

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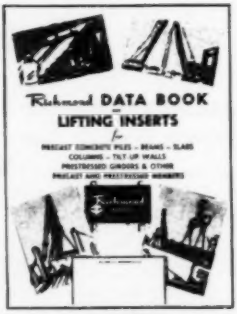
These units are designed with adequate extra strength and for simple operation in lifting and bracing precast concrete components.

Send for your copy of the Richmond Data Book on Lifting Inserts giving complete technical data, dimensions, working loads and ultimate strengths in various strengths of concrete. At the same time ask for your copy of the latest Richmond Handbook showing the complete line of form tying anchorage and accessory devices for concrete construction backed by 47 years in this field.



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Juan Papahiu Kaika..... 2
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Sabri Sami..... 2
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Charles Ream Sargent..... 2
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Robert F. Schoening..... 2
Javier Santolalla Silva..... 2
K. Sitaraman..... 2

Jeno F. Toppler 2
Raul Rios Torres 2
A. M. Tovar 2
Jose R. Perez Valentin 2
Richard D. Walker 2
C. K. Wang 2
J. H. Weaver 2
Richard E. Woodring 2
Lev Zetlin 2

New Members

The Board of Direction approved applications in the following catagories: 66 Individual, 17 Junior, and 2 Student, making a total of 85 new members. Considering losses due to deaths, resignations, retirements, and nonpayment of dues, the total membership on September 1, 1959 was 9983.

Individual

ABRAMOVITZ, MAX, New York, N. Y. (Harrison & Abramovitz)
ARMSTRONG, ORLAN RAY, Dayton Ohio (C. E. Hwy. Engrg., Montgomery County Engrs. Office)
AUXIER, KEITH, Oklahoma City, Okla. (Partner, Auxier-Scott Supply Co.)
BADRENA, FERNANDO, JR., Hato Rey, P. R. (Pres., Badrena y Perez, Inc.)
BALK, KENNETH, University City, Mo. (Cons. Engr.)
BEAM, TERRY, H., Tulsa, Okla. (Bus. Repres., Cement Masons Local Union 690)
BEARD, ARTHUR, Costa Mesa, Calif. (Field Engr., PCA)
BRANDON, JAMES R., Denver, Colo. (Struct. Res. Engr., USBR)
CASSIDY, PAUL G., Villa Park, Ill. (Contr.)
CICO, FRANK, Toronto, Canada (Struct. Dept., John B. Parkins Assocs.)
CLINE, J. E., Tulsa, Okla. (Sales Mgr., Berylex of Okla.)
CLOUSING, T. S., Chicago, Ill. (Gen. Mgr., Insulation Div., M. H. Detrick Co.)
DANI, BACHUBHAI AMBALAL, Ahmedabad, India (Lecturer in Applied Mechanics, L. D. College of Engrg.)
DEVAN, R. V. RANTHI, Orissa, India (Asst. Exec. Engr., Ministry of Irrigation & Power, Govt. of India)
ELLIS, HARRY A., Hamilton, Canada (Constr. Supv., Harry A. Ellis)
ERNST, HENRY P. ERNST, Hinsdale, Ill. (Constr. Spec. Engr.)
ESTENSON, HILMAN A., Iron River, Wis. (Arch.)
FOGARTY, JOHN JOSEPH VINCENT, Ghana, Africa (C. E., Henry J. Kaiser Co.)
FRERIKS, F. G., Pasadena, Calif. (Proj. Engr., Sandberg-Serrell Corp.)
FUENMAYOR C., E. A., Caracas, Venezuela (C. E.)
GARNEAU, HUGHES, Tulsa, Okla. (Pres., Hughes Garneau Co.)
GUILBERT, BERNARD N., Beauharnois, Canada (Coordinator, St. Lawrence Seaway Auth.)
HANLIN, GEORGE B., San Francisco, Calif. (Major, U. S. Army Corps of Engrs.)
HEINZE, L. S., Ottawa, Canada (Mgr., Ottawa Office, Canadian British Engr. Consultants)
HOLTOM, HAROLD THOMAS, Paramount, Calif. (Chf. Engr., Diversified Bldrs., Inc.)
HUMMEL, JOHN W., Louisville, Ky. (Struct. Engr., Arrasmith & Tyler)
JOHNSON, AXEL, Ventura, Calif. (Southern Pacific Milling Co.)
KAPUSCINSKI, STANLEY, Hammond, Ind. (Layout & Detailing Draftsman, Purdue Univ.)
KEITH, GRANT T., Regina, Canada (Pres., Keith Cons. Engrs., Ltd.)
LANGERMAN, PETER STUCKERIS, Durban, South Africa (Mgr., Natal Reg. Office, PCI)
LARON, GEORGE, Willoughby, Australia (Design Draftsman, E. R. Taylor & Assocs.)
LEEGER, S. ERNEST, Bayshore, N. Y. (Constr. Coordinator, Fisher-Malik, Inc.)
L'ESPERANCE, JOSEPH ALFRED ROGER, Camp Valcartier, Canada (Proj. Supv., Defence Constr., (1951) Ltd.)
LI, SIEN YIN, Philadelphia, Pa. (Struct. Designer, Yule, Sticklen, Jordan & McNee, Engrs.)
MALDONADO P., BELISARIO, Santiago de Chile (Tech. Adv. of Cement Factories, Melon-Juan Soldado—Chile)
MARK, CECIL J., Sacramento, Calif. (Area Mgr., Stolte, Inc.)
MAYALL, HENRIQUE CARLOS, Tokyo, Japan (Civil & Mining Engr., Escritorio Tecnico Mayall)
MEAD, FRANK HILLSON, St. Albans, England (Sr. Engr., Oscar Faber & Partners)
NAYLOR, JOHN, Quito, Ecuador (Asst. Resident Engr. R. J. Tipton Associated)
NELSON, GEORGE A., Washington, D. C. (Chf. Research Sect., Veterans Admin.)
OKURA, YASUTARO, Tokyo, Japan (Chf., Est. Sect., Matsui Constr. Co., Ltd.)
ORDUZ, ZALO GOMEZ, Barranquilla, Colombia (C. E., Univ. del Atlantico)
OSAWA, YUTAKA, Tokyo, Japan (Asst., Earthquake Research Inst., Tokyo Univ.)
POLLARA, ROBERT, Port Jervis, N. Y. (Mason Contr.)
POTTER, M. E., Maracaibo, Venezuela (Dist. C. E., Superior Oil Co.)
RINGO, BOYD C., East Lansing, Mich. (Asst. Prof., Civil Engr., Mich. State Univ.)
RIVEROL, DOMINGO ARMAS, Habana, Cuba (C. E., Hormogon Estructal Prefabricado S. A.)
ROSS, DAVID ALLEN, Seattle, Wash. (Asst. Engr., Worthington, Skilling, Helle & Jackson)
SANTAMARIA, RALPH M., Portsmouth, Va. (Struct. Research Engr., & Physicist, Norfolk Naval Shipyard)
SANTIAGO, MIGUEL, College Station, Tex.
SCHWAB, FRANCIS C., JR., Mexico D. F., Mexico (Resident Engr., Hector Mestre y Manuel de la Colina Arquitectes)
SILBER, ENRIQUE, Caracas, Venezuela (Mgr., Industrial Dept., Maquinarias Mendoza)
SIMONSSON, LENNART, Stockholm, Sweden (Dir. in Chf., Statens Provningsanstalt)
SKOGLUND, IVAN RAYMOND, Hartford, Conn. (Design Draftsman, A. J. Macchi Engrs.)
SWEENEY, THOMAS B., Sao Paulo, Brazil (Supv. Engr., Compania Austin)
SZYMORSKI, FRANK L., Brookline, Mass. (Struct. Engr., U. S. Corps of Engrs.)
TAYLOR, CHARLES W. D., Tortola, British Virgin Islands (Supt. of Pub. Wks., P. W. D.)
TUCKER, GORDON, Santa Maria, Calif. (Southern Pacific Milling Co.)
UMERANI, A. RAZZAQUE, Karachi, Pakistan (Struct. Engr.)
VARELA, ALFREDO E. COLIN, Mexico, D. F., Mexico (Subsecretario De Estado, Secretaria De Recursos Hidraulicos)
VENN-BROWN, L. B., North Sydney, Australia (Tech. Cons., Taubmans Cement Compounds Pty., Ltd.)
WARD, WILLIAM V., Houston, Tex. (Supv. Designing Engr., Texas Hwy. Dept.)
WEAVER, JUNIUS V., Lynchburg, Va. (Struct. Design Engr., Montague-Betts Co.)
WEYMARK, W. J., West Vancouver, Canada (Chf., Hydro Studies, Planning Div., B. C. Engrg., Ltd.)
WOOD, ARTHUR G., Denver, Colo. (Sales & Service Engr., Great Western Aggregates, Inc.)
WREDENFORS, WALTER, Stockholm, Sweden (Director, Skanska Cement AB)

Student

ECHEVERRIA, OSCAR, Diaz, Guatemala, Guatemala (Univ. of San Carlos)
TALLEY, WILLIAM L., Detroit, Mich. (Wayne State Univ.)

Junior

ANDERSEN, JAMES ROY, Atlanta, Ga. (Engrg. Design, P & H Engrg. & Constr. Corp.)
BARAJAS, MANUEL ANGEL CASTILLO (C. E.)
CAMINO, RICHARD, Woodside, L. I., N. Y. (Designer, Robins Engineer's Div., Hewitt & Robins, Inc.)
FUERTES, ESTEBAU LUIS, Rio Piedras, P. R. (Engr., Pacadar Prestressed Beams Corp.)
HADJIAN, ASSADOUR, Amman, Jordan (Jr. Design Engr., Development Bd., Kingdom of Jordan)

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RAIL STEEL BARS for concrete reinforcement

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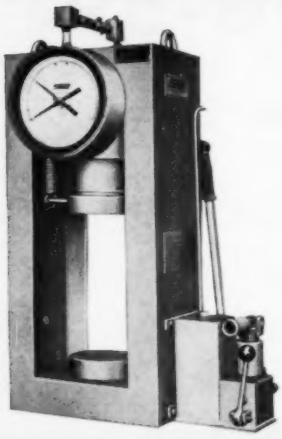
HENSON B., ANTONIO A., Ciudad Trujillo, Dom. Rep. (Cubications & Insp. Engr., Seccion de Estudios De C Anales T Construcciones)
LEHMANN, OSCAR M., Brooklyn, N. Y. (Design Engr., The Preload Co., Inc.)
MASSEY, JOHN A., JR., New York, N. Y. (Struct. Engr., Lev Zetlin, Cons. Engr.)
MOLETON, GERALD P., Laramie, Wyo. (Asst. Engr., Bridge Div., Union Pacific R. R.)
PAULSON, JEROME I., St. Paul, Minn. (Design Draftsman, George Lloyd Levin Engrs.)
PAYNTER, ROBERT LITTELL, Neenah, Wis. (Designer, Frank C. Shattuck, M. F. Siewert & Assocs.)
PORTER, HUGH J., Clearbrook, Canada (Resident Engr., Walker & Webber, Cons. Civil Engrs.)
RODRIGUEZ, BOLIVAR GARCIA, Cidra, P. R. (Engr., Gobierno de Puerto Rico)
SELIM, SAMIR MOUNTAFA, Salt Lake City, Utah (Designer, P. M. Engr.)
SHENOY, DAMODAR S., Fort Bombay, India (C. E., Killick Nixon & Co. Private Ltd.)
SLATER, JOSEPH L., Atlanta, Ga. (Struct. Draftsman, Chastain & Tindel, Inc.)

MODEL FT 20
JOBSITE
CONCRETE TESTER



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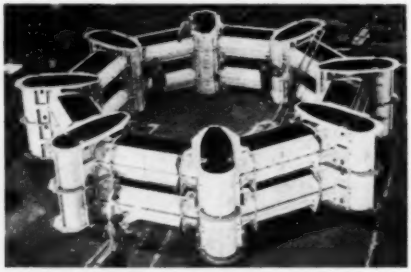
Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Octagonal steel forms for intake structure

An octagonal-shaped unit will be used to concrete an intake trash rack structure at Trinity Dam in northern



California. It marks the first time steel forms have been designed and fabricated for this application.

The forms were made to close tolerances to receive structural gates, and to provide an extremely smooth concrete finish. Each of the eight vertical sections—for forming 3 ft thick piers—is 41½ ft high while the 3 ft thick beam forms between piers are 9 ft long.

The intake structure—approximately 120 ft high—will carry water to a 28 ft diameter diversion tunnel.—Blaw-Knox Co., 300 Sixth Ave., Pittsburgh, Pa.

Fiberboard form void

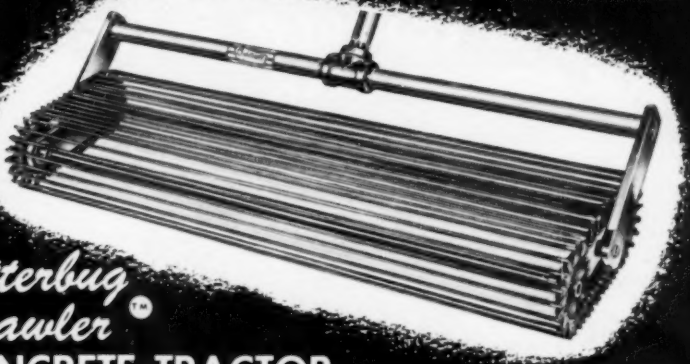
Jay-void, a wax and asphalt fiberboard form for beam and slab construction on grade, is offered by the Lawrence Paper Co. The void is designed to take the place of sand, gravel, and other common fill materials according to the manufacturer. Jay-voids, according to a company spokesman, insure either 4 in. or 6 in. void, depending on the demands of the particular job.—The Lawrence Paper Co., Lawrence, Kan.

Agitator truck

The S & M Manufacturing Co. announce the manufacture of a 6½ cu yd agitator truck for transportation of centrally mixed concrete. The company has manufactured a 4 cu yd model since 1955.

The Agitor is all hydraulic powered requiring no separate engine. Power pump is driven by a standard truck transmission power take off or the S & M Front Power Take-Off is available at additional cost. Hydraulic power turns the agitator, hoists the body, controls the gate, and raises or lowers the chutes. Will handle high or low slump concrete with discharge speeds up to a 1 yard per 10 sec according to manufacturer. Literature available.—The Agitor Division, S & M Mfg. Co., Inc., 2901 West Mill Road, Milwaukee 9, Wis.

New Concept for CONCRETE!



Jitterbug Crawler™ CONCRETE TRACTOR

NO OTHER DEVICE EVER DEVELOPED FOR THE PREPARATION OF POURED CONCRETE SLABS COULD EVER MAKE

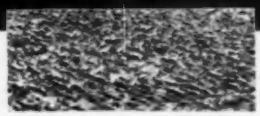
These Proven Claims . . .

- **TRACTORING FLOATS MEDIUM AGGREGATE** Speeds preparation time where stiff slumps are poured, for top quality concrete.
- **TRACTORING MAKES "PASTE" SURFACE . . FAST** Longer contact with mix keeps coarse aggregate down until "fines" rise around and above.
- **TRACTORING ENDS WADING IN THE MIX** Because you 'tamp" a large area with an easy push-pull motion, jitterbugging goes many times faster.
- **TRACTORING "LIBERATES" ONE MAN** Since it's so easy to use, you can use your jitterbug man for many other jobs besides tamping.
- **TRACTORING MAKES NEW TEXTURES POSSIBLE** New textures and new traction surfaces available without costly preparation.

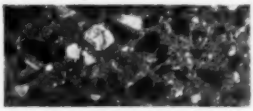
CRAWLING JITTERBUG™

CONCRETE TRACTOR MEANS BETTER CONCRETE . . . IN LESS TIME . . . AT LOWER COST!

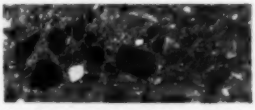
Precision engineered and fully job-proven, the Crawling Jitterbug Concrete Tractor consists of 46 parallel steel rods, each 35" across, operated by gear-driven sprocket chain. Easily operated by one unskilled man, without fluctuation in performance caused by fatigue! Completely assembled, with 12-foot sectional handle, ready to use on the job. Complete, $115.00, FOB Factory in Kansas City. Or write for FREE descriptive booklet on this sensational new machine!



Close-up photo of Tractored surface shows how concrete Tractor's parallel bars press straight down into mix, allowing medium and fine particles to rise around them. No other method leaves surface prepared as shown in this photo!



Sawed cross-section of ordinary slab which was not Tractored shows how the coarse aggregate lies too close to finished surface — ready to peel and spall. Ordinary preparation methods do not let fine and medium particles rise around and above coarse aggregate. Over-working of mix may depress coarse material too far and weaken slab.



Sawed cross-section of Tractored slab shows even distribution of coarse aggregate. Coarse material is held just below surface, to permit rise of perfect paste. But distribution is even, without settling of coarse material to bottom of mix, which would alter monolithic structure of finished concrete.

Goldblatt TOOL COMPANY

FREE Booklet describes new method completely—provides facts and specs on this great machine! Write for your copy, today!

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LONG SPAN VOIDED CONCRETE SLABS FORM BRIDGE DECK

New Highway 67 Bridge, Flat River, Mo.

Contractor: Clark Construction Co.

Engineers: Missouri Highway Department; John Williams, Bridge Engineer, Jim Pasley, Ass't Bridge Engineer.

Voids formed with

SONOCO

SONOVOID®

FIBRE TUBES

reduce slab weight

The voided system of concrete construction permits the design of long span, lighter weight slabs without impairing structural strength.

In the highway bridge illustrated, end spans are 46 feet, and the center spans 56 feet; SONOVOID Fibre Tubes, 15.7" O.D., were used to form voids in the 24½" thick deck slab. By displacing low-working concrete at the neutral axis, SONOVOID Fibre Tubes reduce weight in concrete construction, and save concrete and reinforcing steel.

To speed construction, and for further economy, Seamless SONOTUBE Fibre Forms were also used on this job . . . those of 24" I.D. formed round concrete columns for the abutments and others of 30" I.D. formed the columns in each bent.

Sonoco SONOVOID Fibre Tubes are specifically designed for use in concrete floor and roof slabs, bridge decks, lift slabs, and precast, prestressed concrete piles.

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300-ton portable batching plant

Fairbatch, a 300-ton portable batching plant, is now offered in aggregate or combined aggregate and cement types by the Fairfield Engineering Co. Various arrangements from 2 to 8 compartments are available. Smaller plants are available down to 100-ton.—Paul Wolfert, Standard Products Division, The Fairfield Engineering Co., Marion, Ohio

Curing curtains

Coverlight, neoprene-coated nylon fabric, is being used for concrete curing curtains.

By switching from sheet-metal covers and boxes to fabric, Standard Precast Products Co. of Central Islip, L. I., N. Y., reports savings on labor, storage space, upkeep, and replacement cost. The curtain can be handled by one man, and is steam tight.—Vulcan Rubber Products Division, Reeves Brothers, Inc., 1071 Avenue of the Americas, New York, N. Y.

Lightweight cement hopper

Fruehauf recently introduced their newly designed cement hopper emphasizing weight reduction along with larger capacity. Standard units are 510 cu ft

capacity, but can be obtained in larger capacities as needed. The unit has removable cans and a full 26-in. butterfly gate at the bottom for faster discharge of materials. The lightweight factor increases payload, is easy on the tires, and adds to roadability according to Fruehauf engineers.—Fruehauf Trailer Co., 10942 Harper Ave., Detroit 32, Mich.

Coating material for forms

Nox-Crete form coating is said to react chemically with the concrete mix to prevent bonding between the placed concrete and the inner surfaces of forms and molds. Cost of cleaning forms is essentially eliminated according to the manufacturer.

Nox-Crete form coating may be sprayed, swabbed, or brushed, even at temperatures below freezing. It is said to work equally well on all types of concrete forms and molds, including wood, masonite, and metal. It is claimed to be particularly effective as a bond breaker for tilt-up forms and shell structures.—Industrial Lubricants, Inc., Omaha, Neb.

Portable moisture meter

Exact moisture content of sand, soil, or any bulk material is measured instantly, automatically by a new portable moisture meter according to C & W Sales Co. Moisture content registers on the dial when the probe, connected by insulated cord to the meter, is plunged into the material.—C & W Sales Co., 1490 Franks Lane, Menlo Park, Calif.

Automatic batching control package

The E/E Automatic Batching Control Package for cement and aggregate is designed to offer new or existing plants a low cost "push-button" system according to Engineered Equipment, Inc.

Standardized components can be adapted to fit any batching situation. Two stage cutoff is provided to afford accuracy and control over the batching cycle. Package includes unitized jogging rotary valve or unitized jogging clam shell gate, solenoid control valve, control system, sensing device, and connecting wiring. Sold with or without factory supervised installation.—Engineered Equipment, Inc., Waterloo, Iowa

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Portable compression-tension testing machine

Soiltest have introduced a portable combination compression-tension testing machine which makes it possible to test concrete cylinders or reinforcing bars for strength in a field or laboratory installation. The compression capacity in testing 6 in. diameter standard concrete cylinders is 200,000 lb. The hand operated tester is entirely self contained; no pressure or electrical connections are required.

The machine is also designed for testing specimens in tension up to the 100,000 lb making it possible to test concrete reinforcing bars or small structural steel sections on the job site. Accessory equipment can be added for testing beams in flexure and cubes in compression. Total weight of tester is 500 lb.—Soiltest, Inc., 4711 W. North Ave., Chicago 39, Ill.

Footer form

F. O. Brechel recently patented a form for concrete footing construction consisting of fabricated steel; folding, collapsible corners; offsets; and splices which are used with wood inserts to assemble without nails.

According to Mr. Brechel the forms for an ordinary house or building can be erected in a matter of minutes, and can be disassembled intact and reused.—F. O. Brechel, Bardonner Rd., Route 8, Gibsonia, Pa.

Portable concrete floor grinder

Edco, a portable concrete floor grinder, has been introduced by Equipment Development Co.

Manufacturer states the unit is light enough for two men to carry. Two large, full floating, neoprene-mounted discs, counter rotating, provide about 2 sq ft of grinding area. Can use wet or dry. Available in electric 1½ hp, 110/120 v motor or 3¾ hp, 4 cycle gasoline engine.—Equipment Development Co., 2700 Garfield Ave., Silver Spring, Md.

Nonslip surface

Seeded Emeri-Epox, developed as a modification of Emri-Epox, has been introduced by Walter Maguire Co. The new formulation provides an abrasive, nonslip surface for vehicular and pedestrian traffic according to the company. Traction may be varied, not only by the way in which the plastic is seeded, but also by choice of two different grades of Emery aggregate.

Seeded Emeri-Epox is recommended for special highway, marine, industrial plant, and airport applications.

Application is simple, economical, and quickly completed according to manufacturer. It bonds well to concrete, asphalt, wood, tile, stone, brick, and metal surfaces, offering resistance to chemical attack and mechanical abuse.—Walter Maguire Co., Inc., 60 East 42nd St., New York 17, N. Y.

Ready-to-use protective emulsion

Seaboard Formula 366, a ready-to-use emulsion for sealing and preserving asphalt, concrete, and macadam surfaces, is now being manufactured by Seaboard Plastics Corp.

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LOOKING AHEAD

Jan. 25-29, 1960—Stress Measurement Symposium, Arizona State University, Tempe, Ariz.

Feb. 1-5, 1960—Committee Week, American Society for Testing Materials, Hotel Sherman, Chicago, Ill.

Feb. 15-19, 1960—Annual Convention and Exposition, National Sand and Gravel Association, Conrad Hilton Hotel and Coliseum, Chicago, Ill.

Feb. 22-24, 1960—Annual Convention and Exposition, National Crushed Stone Association, Conrad Hilton Hotel, Chicago, Ill.

Mar. 14-17, 1960—56th Annual Meeting, American Concrete Institute, Commodore Hotel, New York, N. Y.

or mixing before application. May be applied with soft floor brush, rubber squeegee or spray.—Seaboard Plastics Corp., Irvington, N. J.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

PORT COLUMBUS TERMINAL—Describes the use of precast concrete floor and roof units in the main terminal building of the Port Columbus Airport in Columbus, Ohio. Photographs and detail drawings illustrate how separate sets of the hollow cells in the concrete slabs were used as raceways for three complete underfloor signal, electrical, and telephone distribution systems.—The Flexicore Co., Inc., 1932 E. Monument Ave., Dayton 1, Ohio

MONOLITHIC FLOORING MATERIALS (Bulletin 3-4)—Folder describes Rezklad monolithic flooring materials, which manufacturer claims offer a new concept in floor protection—an economical one-layer corrosion-proof flooring system designed specifically for application to horizontal concrete surfaces. Tables show physical properties and chemical resistance of the materials.—Atlas Mineral Products Co., Mertztown, Pa.

AMERICAN LOOK—Color motion picture previewing things to come from the drawing boards of prominent architects and designers. Free for group showing.—The Jam Handy Organization, 2821 East Grand Blvd., Detroit 11, Mich.

BIN VIBRATOR CATALOG—Illustrated 12-page catalog on "pulsating-magnet" electric vibrators to aid flow of bulk materials and bulk parts from bins, hoppers, and chutes. Gives data for 14 standard vibrators, ranging from diminutive model for materials of less than 1 cu ft to the V-600 model, capable of vibrating bins with capacities weighing in excess of 150 tons.—Syntron Co., 1471 Lexington Ave., Homer City, Pa.

PENNSALT BULLETIN S-145—Technical bulletin containing a summary of performance of Pennsalt Curing Agents S-1 and S-2. Gives data on physical properties, tensile strength, hardness, compression strength, heat and chemical resistance, heat distortion, applications, epoxy-polysulfide combinations, toxicity, and handling information.—Pennsalt Chemicals Corp., 3 Penn Center, Philadelphia 2, Pa.

REX TILT-UP MIXER (Bulletin 59209)—Presents complete information on Rex Tilt-Up Mixers for ready-mixed concrete plants, Porto-Plants, and central-mix paving. Illustrates installation, design, and mixing advantages of the Tilt-Up Mixer.—Chain Belt Co., Sales Promotion Dept., Milwaukee 1, Wis.

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GUIDE TO CORRECT CYLINDER CASTING (RM-48)—Wall chart type guide showing approved concrete cylinder casting procedures including selection of molds; correct sample taking; and filling, handling, and curing of cylinders. Suitable for posting at job sites where testing is performed.—The Master Builders Co., Cleveland 3, Ohio

SYNTRON CATALOG 596—New condensed catalog of materials handling equipment, parts handling equipment, power rectification equipment, mechanical shaft seals, paper joggers and portable power tools. The 68-page catalog presents descriptions, data, and specifications on all Syntron products. Also gives latest listing of Syntron representatives.—Syntron Co., Homer City, Pa.

CRACKS IN CONCRETE—Eight-page folder shows examples of eight common types of cracks, explaining the cause of each. In addition, there are sketches of magnified sections of concrete showing six less common types of cracks, indicating their probable causes. A short discussion of cracking and of surface imperfections is also included. The bulletin is the fourth in a series of "Craftsmanship in Concrete" service bulletins prepared by Alpha.—Alpha Portland Cement Co., 15 S. Third St., Easton, Pa.

HYDRO-SCREEN WASHING PLANTS (Bulletin 690)—Colorful, 8-page bulletin describing Pioneer's new line of Hydro-Screen Washing Plants. Drawing of the basic plant is supplemented by drawings of several different models of the plant designed to meet specific aggregates washing problems. Specifications, field photos, and complete detailed information on Hydro-Screen plant line is included.—Pioneer Engineering, 3200 Como Ave., S. E., Minneapolis 14, Minn.

REX PORTO-PLANTS (Bulletin 59210)—Carries complete information on Rex Porto-Plant Models 60 and 125 and their components, including: portable aggregate bins, portable cement storage bins, batching equipment, portable belt conveyors, screw conveyors, and advanced control systems.—Chain Belt Co., Sales Promotion Dept., Milwaukee 1, Wis.

RADIANT HEATING SLAB CONSTRUCTION—Illustrated bulletin designed to aid architects, engineers, heating and piping contractors, and others concerned with accepted installation procedures for radiant heating systems. Contains sketches of various types of radiant heating slab construction. Each sketch shows position of wrought iron pipe coils in the concrete slab as well as relationship of coils and slab to structural features of on grade and above grade construction.—A. M. Byers Co., Box 1076, Pittsburgh 30, Pa.

PRE-BILT SECTION BELT CONVEYORS (Book 2779)—Forty-page handbook on the use and selection of standardized Pre-Bilt sectional belt conveyors. Illustrations show installation applications in a variety of industries and typical layouts. A total of 22 components and accessories are listed in the handbook and also detailed engineering and selection data.—Link-Belt Co., Dept. PR, Prudential Plaza, Chicago 1, Ill.

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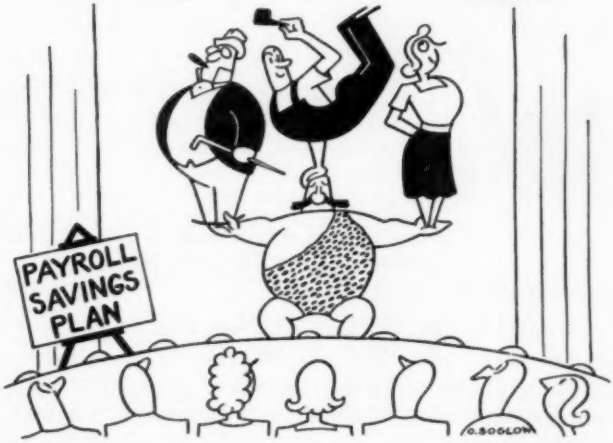

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Goldblatt Tool Company 19

Jackson & Moreland, Inc. 23

Lehigh Portland Cement Company 3

Marathon, Division of American Can Company 12

Master Builders Company, The; Division of American-Marietta Co. 4

Rail Steel Bar Association 17

Richmond Screw Anchor Co., Inc. 14

Sika Chemical Corporation ..... iv (flyleaf)

Soiltest, Inc. 9

Solvay Process Division, Allied Chemical 24

Sonoco Products Company 20

Water Seals, Inc. 25

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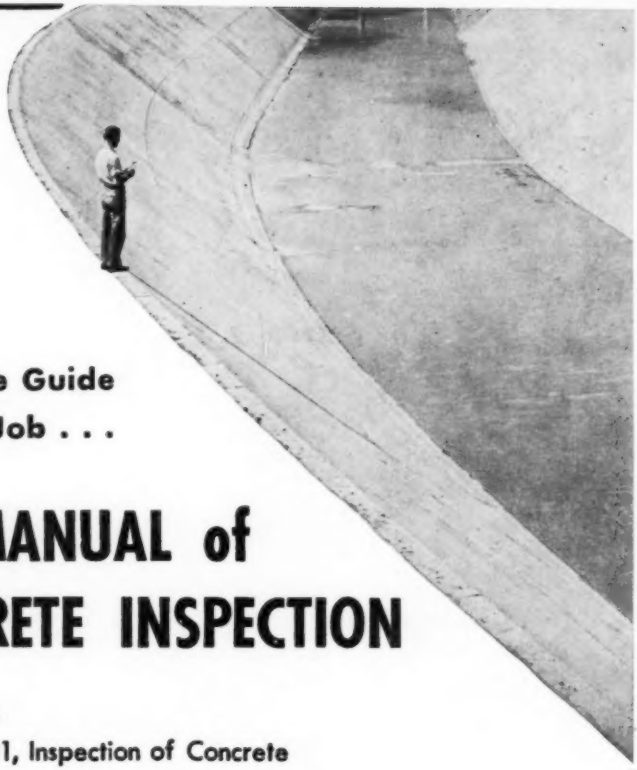

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THIS MONTH

Papers and Reports **273-338**

56-19 **Ultimate Loads and Deflections from Limit Design of Continuous Structural Concrete** .....
G. C. ERNST and A. R. RIVELAND **273**

56-20 **Problems and Performance of Precast Concrete Wall Panels**
VICTOR F. LEABU **287**

56-21 **Effect of Admixtures on Electrolytic Corrosion of Steel Bars in Reinforced Concrete** .....
YASUO KONDO, AKIHIKO TAKEDA, and SETSUJI HIDESHIMA **299**

56-22 **Comparison of Measured and Calculated Stiffnesses for Beams Reinforced in Tension Only** ..... BILL G. EPPES **313**

56-23 **Rheological Behavior of Hardened Cement Paste Under Low Stresses** ..... J. GLUCKLICH **327**

Concrete Briefs **339-340**

Current Reviews **341-360**

News Letter **1-32**

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2 AL 5 DE NOVIEMBRE